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Methods of Handling and Placing Concrete at Shasta Dam*

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Member American Concrete Institute

SYNOPSIS

Describes briefly equipment and methods employed by contractors in handling and placing 6,500,000 cubic yards of concrete in construction of Shasta dam and power plant.

INTRODUCTION

Shasta dam and power plant are being constructed under the direction of the Bureau of Reclamation by Pacific Constructors, Inc. The dam, of concrete gravity type, 3500 ft. long at the crest and 602 ft. high, consists of a straight overflow spillway section and slightly curved abutment sections. Flow of water over the spillway will be regulated by three 110 x 28-ft. drum gates mounted on the crest between piers which will also serve as supports for a bridge connecting roadways of the abutment sections. Variable height training walls 6 ft. thick extending from the spillway crest to the spillway apron will confine overflow water to the spillway section. A stilling pool 40 ft. deep at normal flow, is provided by excavation below the river bed for the spillway apron.

The power plant, a reinforced concrete structure, is on the right bank of the river approximately 800 ft. downstream from the dam. It will house five 75,000-KVA generators driven by five 103,000-horsepower turbines and two 2300-KVA station service units.

The most interesting and unique feature of the contractor's construction layout (Fig. 1) is the distribution of concrete over the job by a system of seven radial cableways, all operated from a single stationary structural steel head tower 460-ft. high.‡ Traveling tail towers on both abutments operate on radial tracks and provide complete coverage of the dam, spillway apron and power house.

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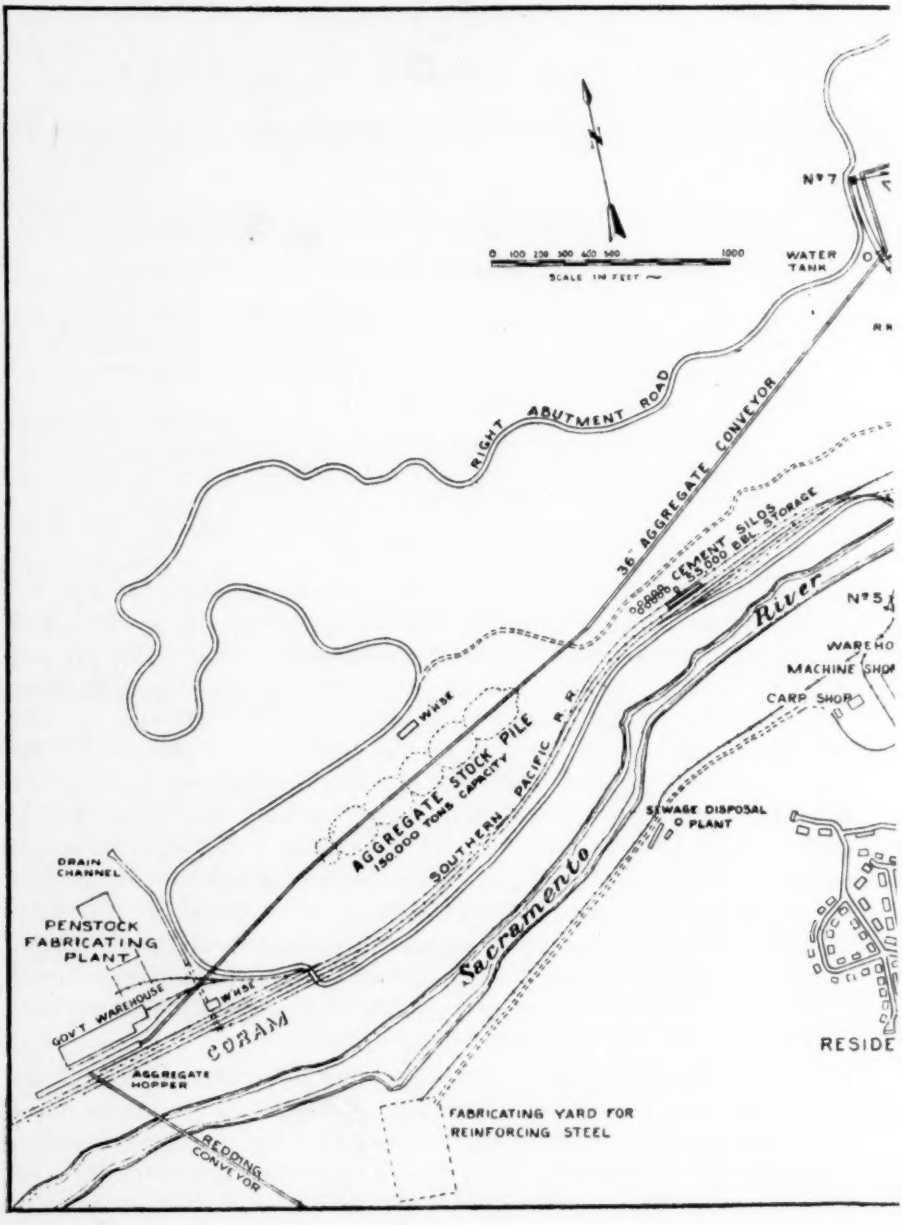
‡*Engineering News-Record*, July 4, 1940.


N
0 100 200 300 400 500 1000
SCALE IN FEET
WATER TANK
No 7
RIGHT ABUTMENT ROAD
36" AGGREGATE CONVEYOR
CEMENT SILOS
55,000 BBL. STORAGE
River
No 5
WAREHO
MACHINE SHOP
CARP SHOP
WHSE
AGGREGATE STOCK PILE
150,000 TONS CAPACITY
SOUTHERN PACIFIC R.R.
SEWAGE DISPOSAL PLANT
DRAIN CHANNEL
PENSTOCK FABRICATING PLANT
WHSE
GOV'T WAREHOUSE
CORAM
Sacramento
AGGREGATE HOPPER
BEDDING CONVEYOR
FABRICATING YARD FOR REINFORCING STEEL
RESIDE

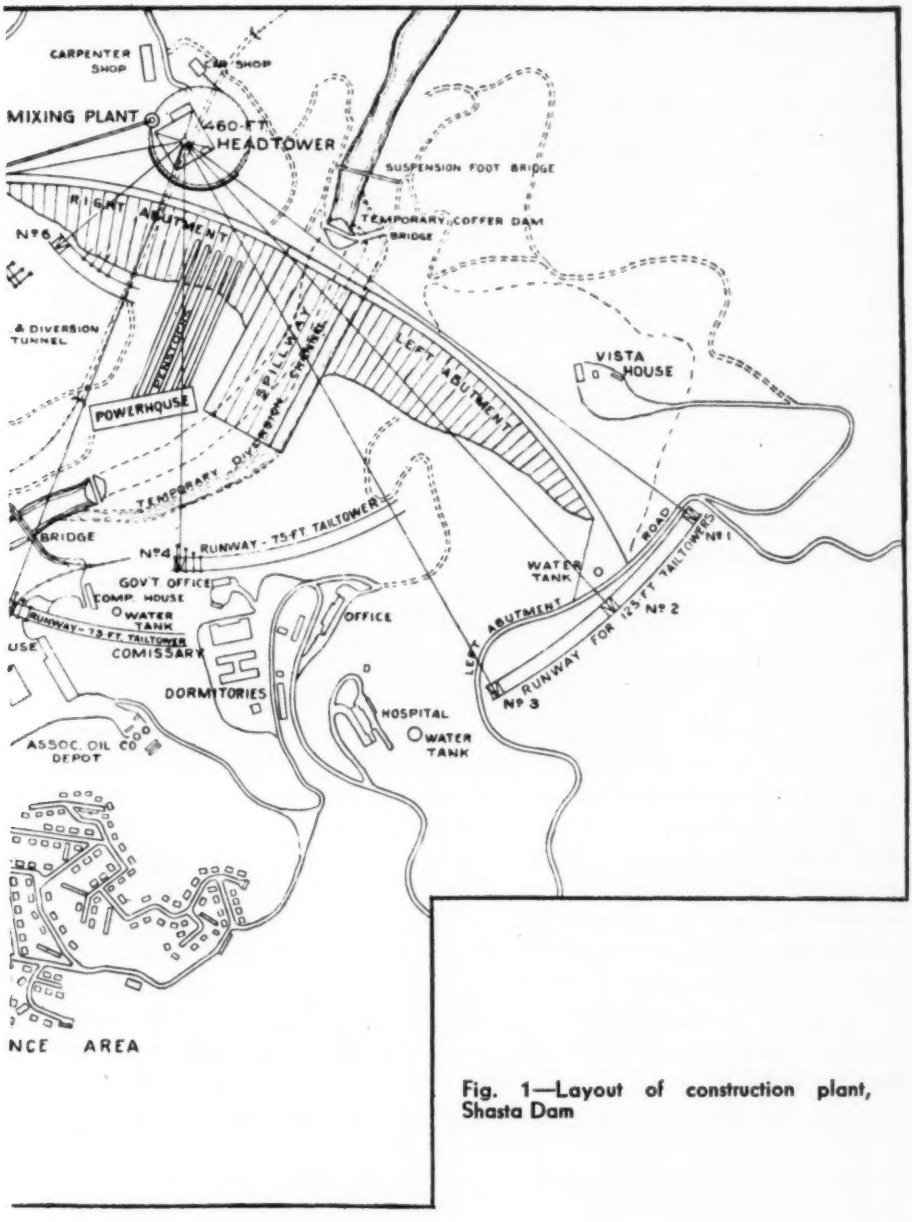


Fig. 1—Layout of construction plant, Shasta Dam

MATERIALS

Aggregates are obtained from a natural river deposit approximately ten miles downstream from the dam. Excavation and processing are done under contract between the Bureau of Reclamation and Columbia Construction Co., Inc. Aggregate is separated into sand and five sizes of gravel; namely, 6 to 3 in., 3 to 1½ in., 1½ to ¾ in. and ¾-in. to No. 4.

Eleven hundred tons of aggregate per hour are transported from the processing plant to hoppers near the dam, a distance of 9.6 miles, by 26 flights of a 36-in. belt conveyor.* Another system of 14 belt conveyors transports aggregate an additional 1.2 miles from hoppers to stockpiles and thence around the right abutment to the mixing plant. (Fig. 1).

Cement, a low heat type, is manufactured by the Permanente Cement Corp., San Jose, Calif., and transported by railroad to the dam. Two Fuller-Kenyon cement pumps capable of unloading 900 bbl. of cement per hour are used to unload cement from railroad cars and blow it into silos of 55,000-bbl. capacity. Cement is pumped or blown from the storage silos to the mixing plant through a 10-in. pipe line. This cement pumping line is 3300 ft. long and includes a 280-ft. net lift.

PRODUCTION

The mixing plant, a steel structure, designed for a capacity of 10,000 cu. yd. per day, has automatic batching equipment and five 4-cu. yd. mixers. Aggregate and cement storage is provided in the upper sections for eight hours of operation.

Batching equipment was designed by C. S. Johnson Co. and is similar to that in other modern concrete plants with push button and interlocking controls making the operations almost automatic. All materials are measured by weight and weights recorded on a combined graph recorder. Concrete consistency is controlled by consistency meters and slump tests. Consistency meters are based on the principle that the center of gravity of the mixers changes with changes in concrete consistency and by means of a system of levers and knife edges attached to the supports of the mixer, these changes are measured and recorded. Meters are calibrated and continuously checked by slump tests.

Mass concrete placed in the dam has the following mix proportions:
Mix—1:2.5:7.1
W/c—.60 by weight.
Slump—2″.
Cement content—1.00 bbl. per yd.
Sand fineness modulus—2.9

*ACI JOURNAL, Feb. 1942; *Proceedings* Vol. 38, p. 329: "The Effect of Belt Transportation on Concrete Aggregate Grading," by Gordon L. Williams.

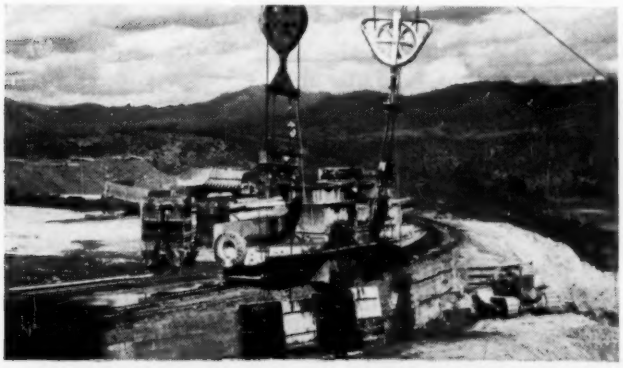


Fig. 2—Discharge of concrete from hopper cars to cableway bucket

Fig. 3—Discharge of 8-cu.yd. concrete bucket

Operation of the two hoist lines used to open and close gates can be seen at the end of the bucket

Compressive strength of 6 x 12-in. control cylinders after 28 days' standard curing is 3000 psi. and after 90 days it is 5000 psi.

TRANSPORTATION

Mixers are around, and discharge into, a circular cone-type hopper capable of storing 16 cu. yd. of concrete. The hopper discharges into special cars that operate on a circular track, 210-ft. radius, which extends under the plant and around the base of the head tower. These cars were job assembled, all steel, electric driven, and carry two 8-cu. yd. hoppers with large quick opening discharge gates in the bottom.

A landing is provided for cableway buckets directly below the hopper car tracks. Buckets are spotted on the landing by the cableway operator who, from his position in the head tower, can observe all operations below. Hopper cars discharge concrete directly into the bucket, the bucket remaining attached to the cableway. Fig. 2 shows the transfer of 8 cu. yd. of concrete from the hopper cars to the cableway bucket.

Concrete buckets are hoisted from the bucket landing and transported via the highline to the point of placement. They are spotted at the point of placement by a bell boy located on the forms who is in telephone communication with the cableway operator at all times. The bucket is rectangular with two side-hinged gates forming the bottom. Opening and closing of the gates is controlled by manipulation of two hoist lines, one attached to the top of the bucket and one to the gates in the bottom. By close control of these two lines, it is possible to regulate the discharge as desired. Safety latches on the sides have to be unfastened by workmen on the block before the bucket can be dumped. Fig. 3 shows the cableway bucket discharging 8 cu. yd. of concrete in a 50 x 50-ft. block on the dam. The bucket was designed for this job by contractor's engineers.

For placing concrete in the power house and in confined areas of the dam where wetter concrete containing smaller size aggregate is desired, a circular cone-type bucket is used with a hand-operated gate in the bottom. In large, massive, reinforced sections, concrete is discharged from the bucket directly into the forms. In more restricted areas a staging is constructed above reinforcement bars. Hoppers and elephant trunks are then located at approximately 10-foot intervals over the entire area of the section and concrete dropped from the bucket through the hoppers and trunks into the forms.

PLACING

Preceding the placement of concrete, all approximately horizontal rock surfaces and construction joints are covered with a one-half inch layer of mortar, which is worked into all irregularities with stiff wire brooms. Mortar has the same proportions of sand and cement and slightly lower W/C ratio as the concrete.

Concrete in the dam is placed in four horizontal layers per 5-ft. lift. Entire contents of the cableway bucket (8 cu. yd.) are deposited in one place and compacted with three large electric internal vibrators manufactured by the Chicago Pneumatic Vibrator Co. (Fig. 3, 4 and 5.) Occasionally it is necessary to deposit smaller quantities at a time in corners and in confined areas around reinforcement. Large cobbles that become separated from the mass are distributed by shovels over the pile as the concrete is being vibrated. Placing crews consist of a foreman, one laborer, and six vibrator men. The top surface of each 5-ft. lift is left relatively smooth by the vibrator and no attempt is made to puddle rocks into the surface.

Numerous delays are caused by heavy rains in winter months. However, there are many days in which rainfall is light and concrete place-

Fig. 4 — Eight cubic yards of concrete immediately after dumping from cableway bucket and before vibration

Fig. 5—The same concrete shown in Figure 4 as the vibration nears completion

ment can progress satisfactorily. (A light rain may be described as one in which the precipitation is not in excess of .05 inch per hour). When concrete is placed under these conditions, the mortar layer is spread in small quantities and covered with concrete as soon as possible. Mortar is mixed somewhat drier than that ordinarily used to compensate for water mixed in at the block. Instead of placing concrete in horizontal layers as outlined above, it is placed on a flat slope, from downstream to upstream. Each bucket of concrete when placed is vibrated to a smooth surface to facilitate drainage of water down the slope. Free water is blown off the surface immediately prior to the dumping of each bucket of concrete and is collected in a sump maintained near the upstream end of the block. Water is removed from the sump by means of siphons, pumps, shovels, sponges, etc. Some cement is washed from the surface and is pumped out of the block along with free water. The actual quantity of cement lost is unknown, but the effect is not considered detrimental. When placing concrete during rainfall special care is taken to weld succeeding layers together by deep penetration of the vibrators.

CONSTRUCTION JOINT CLEAN-UP METHODS

Methods used in curing and cleaning construction joints will be treated in detail by a later article when more data are available. Specifications called for the placement of 2 in. of wet sand on all horizontal construction joints as soon as concrete had hardened sufficiently to withstand traffic in placing. Immediately before the next lift of concrete was placed, sand was removed and the surface thoroughly washed with air and water jets. This procedure was followed from the beginning of concrete placement until March 1941 with apparent satisfactory results. Cost of sand in this vicinity, together with high cost of handling sand, caused this method of joint treatment to be very costly. The contractor, since March 1941 has been sandblasting all joints in lieu of the sand cure method. Water sprays provide curing of all surfaces.

The construction of Shasta Dam is being supervised for the Bureau of Reclamation by Ralph Lowry, Construction Engineer; Grant Bloodgood, Field Engineer; and W. A. Dexheimer, Chief Inspector. Frank T. Crowe is General Superintendent for Pacific Constructors, Inc.

Discussion of this paper should reach the ACI Secretary in triplicate by March 1, 1943 for publication in the JOURNAL for June, 1943

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Effect of Grinding in the Large Mixers on Aggregate Grading at Hiwassee Dam*

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SYNOPSIS

Preliminary investigations revealed that changes in grading due to grinding in the mixers would be a major consideration in the concrete control. Crushing tests gave the first indication of the instability of "graywacke" and laboratory mixing tests were made to determine the relative grinding characteristics of "graywacke" and dolomite.

Preliminary tests served only to confirm the existence of the problem, and field investigations were begun concurrently with concrete placing to determine amount and character of grinding taking place in the large field mixers. "Grab" samples were analyzed periodically as a basis for adjusting "ingoing" grading to compensate for changes in grading due to grinding in order to get a desired aggregate grading in the mixed concrete. Later full batches were analyzed as an additional check on changes in grading due to grinding. "Split batch" charging of the "face" concrete was adopted after job trials indicated that the amount of grinding could be reduced by withholding coarse rock and cobbles from the mixers until one half the mixing time had elapsed.

Variable grinding during mixing made production of concrete of uniform quality more difficult. Several conclusions in regard to grinding are given at the end of the paper.

Hiwassee Dam is the second large tributary dam constructed by the TVA in the unified program of the Tennessee Valley development. It is in Western North Carolina on the Hiwassee River. The dam is a straight-gravity concrete structure approximately 300 feet high with an overflow spillway section in the natural river channel. The dam and powerhouse contain approximately 800,000 cubic yards of concrete.

The dam is in a region where practically no natural concrete aggregate materials exist, and the isolated location made transportation rates from the nearby natural deposits prohibitive. This led to the consider-

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Fig. 1

ation and subsequent adoption for use of a crushed aggregate which could be produced at the job by quarrying, crushing, and processing the material available at the site, commonly known as "graywacke." The "graywacke," more correctly classified according to the geologist as "quartz mica gneiss," is characterized by its high silica content (average about 75 per cent), its relatively high density, (s.g. 2.74), and its variation in physical appearance. In physical appearance it varies from a fine-grained dark-colored rock to a coarse-grained light-colored rock with innumerable intermediate combinations of grain size and color (Fig. 1). Generally speaking the material was roughly placed in three classifications according to grain size only. This was not a precise classification since even the so-called coarse-grained material had a mixture of particles ranging down to very fine material. A preliminary estimate of the quarry selected, indicated that about 10 per cent of the material was fine grained, about 70 per cent medium grained and about 20 per cent coarse grained.

The location of a source of suitable concrete aggregate for any large hydraulic structure is a major problem. A problem which is usually neglected in preliminary concrete investigations is that of the effect of grinding action in the mixing process on the grading of the aggregate in the mixed concrete. Depending on the material under consideration, the importance of the problem of "grinding" is directly related to the amount

and character of the grinding which takes place. It is entirely conceivable that in some instances beneficial effects have accrued to the aggregate grading as a result of grinding. Particularly is this possible where natural sand which is deficient in fines is used in "mass" concrete with a low cement factor.

PRELIMINARY INVESTIGATIONS

During preliminary investigations it was discovered that one of the characteristics of "graywacke" was its instability during handling, and its general tendency to break down producing an excess of fine material. All grades of the rock exhibited this tendency in some degree; the grain size of any particular classification influencing to a great degree the manner in which it broke down, and the minimum particle size formed by the process. For instance, the fine-grained dark-colored "graywacke" tended to break into very fine grains; whereas, the coarse-grained light-colored "graywacke" tended to break into material about the original size of the grains in the material. Another tendency exhibited was that of breaking down to a minimum grain size and no further. That is to say, if a certain classification tended to form a minimum grain size which would pass a 48-mesh screen and be retained on a 100-mesh screen it was very difficult to reduce this grain size to minus 100-mesh material. In this same connection there seemed to be a minimum grain size between the 200-mesh and 325-mesh screen sizes, below which very little material was produced. All grains larger than the apparent minimum for a particular classification were unstable to a degree, and in handling and mixing these tended to break down further. It seems almost incredible that throughout the job the minus 100-mesh material in the sand increased an average of two per cent between the stacker at the stockpile and the batchers in the mixing plant. The finished sand, as stockpiled, contained about ten per cent minus 100-mesh material, and as introduced into the batcher contained about 12 per cent minus 100-mesh material. Breakage in the coarse aggregate sizes was appreciable but not particularly serious, except when it became necessary to "bulldoze" in the stockpiles. The aggregates were divided into five nominal size groups designated as cobbles (minus 6-in. plus 3-in.), coarse rock (minus 3-in. plus 1½-in.), medium rock (minus 1½-in. plus ¾-in.), fine rock (minus ¾-in. plus No. 4), and, sand (minus No. 4). The four coarse aggregates were washed over a rinsing screen located between the stockpile and the mixing plant bins to remove the fines which had accumulated due to breakage in handling. The coarse aggregates in the mixing plant bins, therefore, contained no fines except those resulting from a very small amount of breakage in the final fall into the mixing plant bins.

Preliminary crushing tests gave the first indication of the tendency to breakdown and produce an excess of fine material. Before any tests on grinding during mixing were considered, a series of crushing tests were made to determine the effect of grain size on breakage characteristics. Core samples representing the three classifications, according to grain size, were crushed in a laboratory crusher under as nearly identical conditions as possible. The results of this crushing test are shown in Table 1. Additional tests similarly conducted, but with the materials further subdivided according to shade, confirmed the results of the first tests.

TABLE 1

INDIVIDUAL PERCENT RETAINED

Screen Size	Fine Grained Graywacke	Medium Grained Graywacke	Coarse Grained Graywacke
½-in.	0.0	0.0	0.0
⅜-in.	0.7	0.8	0.6
3 Mesh	12.5	11.3	11.0
4 "	27.8	29.5	29.5
8 "	21.7	26.2	26.6
14 "	8.8	8.9	9.4
28 "	5.3	4.5	4.7
48 "	3.3	3.1	3.4
100 "	3.2	4.2	4.5
Pan	16.7	11.5	10.3

For purposes of comparison, a test was run in which a sample of "graywacke" aggregate and a sample of "dolomite" aggregate, each with identical gradings, was mixed dry in a laboratory mixer, for stated intervals, and screened at the end of each interval to determine the effect on the grading (Fig. 2). Dolomite aggregate used in the test was from the crushed aggregate used in the concrete for Norris Dam. The "dolomite" aggregate had demonstrated in previous unrelated tests remarkable toughness and hardness, and it can be said that in this test the "graywacke" aggregate was compared with a very high grade crushed aggregate. Norris dolomite showed less resistance to wear than "graywacke" but greater toughness. Tests conducted by the Materials Laboratory of the Tennessee Valley Authority in Knoxville showed for the Norris dolomite wear of 4.3 per cent as compared to 3.2 per cent for "graywacke" (ASTM D2-33); whereas for the Norris dolomite toughness was 7.5 as compared to 6.5 for "graywacke" (ASTM D3-18). Test data is meager in regard to wear and toughness, but the toughness relationship indicated by the Materials Laboratory test was borne out in the dry mixing test of the two materials. As can be seen from Fig. 2 this test indicated two things very clearly: first, the breakage in both materials was larger in the smaller sizes, decreasing in amount in the larger material

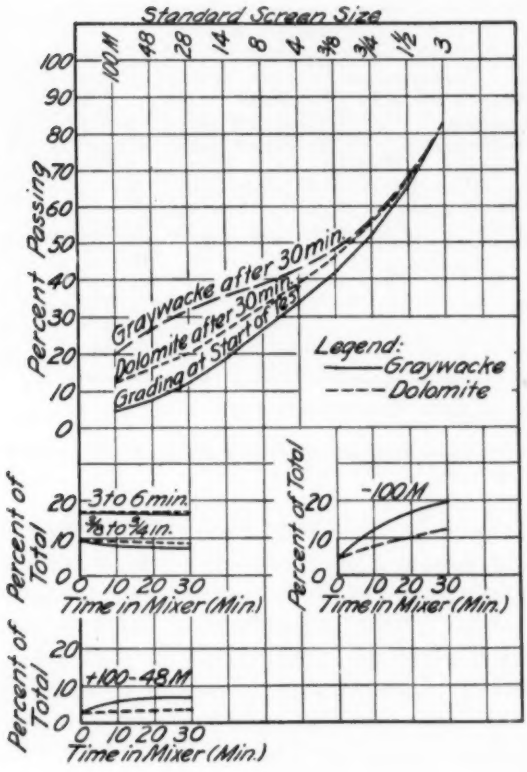

Fig. 2—Aggregate breakage tests

with no appreciable breakage in the cobble size (−6 + 3-in.). This indicated a "ball-mill" action in which the smaller particles are crushed between the large particles. The second indication was that in the "graywacke" the breakage is more rapid at the beginning of the grinding period than later. The first indication that the material had a tendency to break down to a minimum grain size and no further was indicated in the curves showing "per cent of total" plotted against "time in mixer" for the minus 48-mesh plus 100-mesh, and minus 100-mesh (Fig. 2). Note that the slope of the curves becomes flatter with increased time in the mixer.

Another series of tests was made for the purpose of determining the effect of mixing time on change in consistency. In this series three mixing tests were made, using in the first test dolomite coarse and fine aggregate; in the second test, graywacke coarse and fine aggregate; and, in the third test, graywacke coarse aggregate with dolomite fine aggregate. In these tests grading was identical and cement content equal, but enough mixing water was added to get approximately equal slumps at the end of

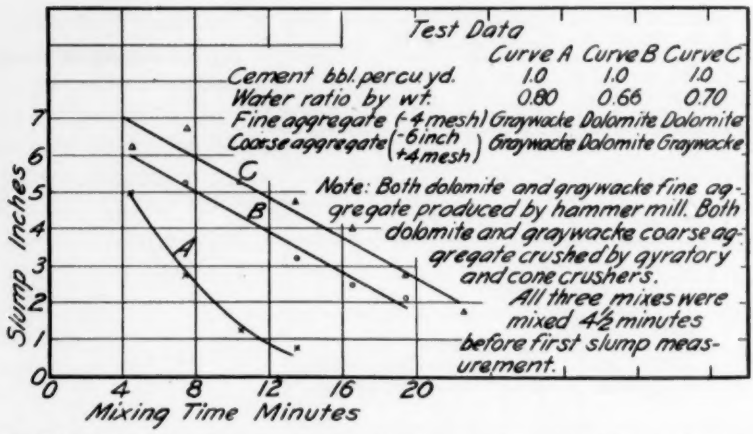

Fig. 3—Effect of mixing time on slumps of concrete mixes containing various types of aggregate

three minutes of mixing in a 3½-cu. ft. laboratory mixer. Each test was continued, after the first three minute mixing interval, for stated intervals, and a slump test taken at the end of each interval. Fig. 3 shows graphically the results of these tests. It will be noted that slump decreases rapidly with increased mixing time for the "all graywacke." The effect of mixing time on change in slump is about the same whether all dolomite was used or whether graywacke coarse and dolomite fine aggregate was used, demonstrating that breakage in the coarser sizes is not appreciable. Individual slump measurements were erratic, but the trends are very definite. The three curves considered together confirm results of the dry mixing test which indicated appreciable breakage in the graywacke fine aggregate, and, also, the slope of the "all graywacke" curve indicated again the tendency for grinding to be very rapid at the beginning and decrease with mixing time.

FIELD INVESTIGATIONS

All of these tests were valuable as indicating the general characteristics of the material, but they were not expected to be anything more. The effect on changes in grading due to grinding in the large mixers of the central mixing plant was still a matter to be investigated in the field. Many field conditions can be similated in the laboratory, and in recent years model testing has helped to solve many problems, but if it is not impossible it is at least impractical to predict by tests in a laboratory mixer with 3½-cu. ft. capacity the grinding which might take place in a 3-cu. yd. capacity field mixer.

During the first four months of concreting, "grab" samples weighing about 100 to 125 lb. were taken at the mixer discharge and wet screened

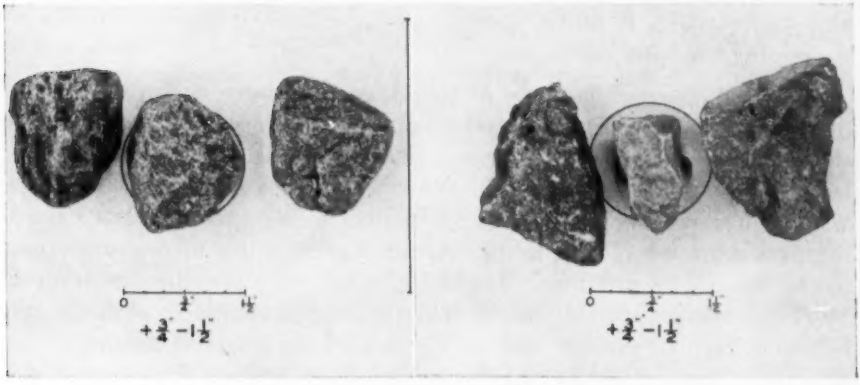

Fig. 4—Comparison of graywacke and dolomite aggregates

(Note rounded edges of graywacke due to wear in handling, and angular edges of dolomite which withstood handling without appreciable wear at the edges.)

to determine the grading of the aggregate after mixing. A large number of these tests were made and an average grading of aggregate in the mixed concrete determined from them. This was compared with the average theoretical combined aggregate grading going into the mix as determined from analyses of samples of aggregate taken over the same period. Table 2 shows a comparison of results for the four months' period.

TABLE 2—COMPARISON OF SAND GRADING IN MASS CONCRETE BEFORE AND AFTER MIXING

	Indiviuual Percent Retained							
Screen Size	8	14	28	48	100	Pan	FM	No. of Tests
*Before Mixing	21.5	20.7	15.4	13.4	13.6	15.4	2.77	163
**After Mixing	19.0	14.2	11.0	11.9	12.8	31.1	2.21	16

*Average of composite mixing plant sand samples.
**Average of "washdown" tests on grab samples taken at the mixer discharge.

The trend of change in grading due to grinding during mixing was determined from these tests, and adjustments in the "ingoing" combined aggregate grading were made, as far as possible, to compensate for the change. These tests revealed an increase in total sand in the mix, and a large increase in minus 100-mesh material in the sand. A reduction in total sand going into the mix was effected in order to partly compensate for this. In this same connection the minus 100-mesh material in the finished sand was finally reduced from about 15 or 16 per cent down to about 10 per cent by wet classifying. The average fineness of the sand,

also increased by grinding during mixing, suggested the advisability of decreasing the sand content.

The limitations of the type of test described above were realized and duly considered in using the test results for making compensations. It should be recognized, that because in the past usually no account has been taken of changes in grading due to grinding in the mixers, there was no precedent of past experience to follow as a guide in either arriving at a most desirable grading, or, for making adjustments in grading to compensate for grinding. The latter, of course, was subject to analysis by the methods outlined above. The problem in this case, however, did not lend itself to precise analysis because of the variable nature of the material. The experience at Hiwassee seems to indicate that some of the differences of opinion which now exist about desirable aggregate gradings, particularly sand gradings, might not now exist if the observers had taken into account changes in grading due to grinding. It is also the usual case that desired gradings determined by laboratory experiment have to be adjusted in the field. This may be due in many instances to grading changes (which are adjusted for but not recognized) caused by grinding in the large field mixers.

Realizing the limitations of the tests on the small "grab" samples which served as the basis for the original mix adjustment, it was decided to analyze some full 2½ cu. yd. batches. In order to analyze a 2½-cu. yd. batch it was of course necessary to omit the use of cement since the job of screening a batch containing 2½ cu. yds. is about a week's work, whereas the 100-lb. samples previously taken could be finished in an hour or less, which allowed removal of the cement before it hardened. In place of the cement, minus 100-mesh classifier fines were substituted by absolute volume in order to simulate as nearly as possible the mixing action when cement was used. Six 2½-cu. yd. batches were analyzed, three of which were charged and mixed for 2½ minutes in the regular manner, and three of which were charged on what was designated "split batch" charging and mixed for a total of 2½ minutes.

The "split batch" charging cycle was accomplished in two parts: first, the water, cement (or minus 100-mesh fines substitution), sand, and two smaller sizes of rock were charged into the mixer and mixed for 1¼ minutes; and, second, the coarse rock and cobbles were charged and mixing continued for another 1¼ minutes. The objective was to reduce the amount of grinding, since the grinding, it was assumed, was largely a result of "ball-mill" action with the coarse aggregate grinding the sand. This method of charging and mixing was adopted for "face" concrete during the latter half of the job. In practice the effect of this method of charging was evidenced by a small but definite decrease in

water requirement for concrete of equal consistency (slump about 1½ in.) as compared with concrete produced by the usual method of charging all the materials at one time. The variable nature of the material made it difficult to determine precisely the amount of the difference, but over an extended period it seemed to average about 5 lb. per cu. yd.

The full-scale tests were conducted as follows: A quantity of fines (−100-mesh sand) equal in absolute volume to the absolute volume of cement was prepared in advance. At the time of the test the aggregates were weighed as usual in the central mixing plant. As the aggregates were weighed, representative samples of each were taken. From the analyses of the samples and the known weights of aggregate, a combined aggregate grading was computed. The aggregates and the substitute for cement (the −100 mesh fines) were charged as desired and mixed. The entire batch was then analyzed in the course of which it was determined what amount of −100 mesh material was contained in the batch. The amount of −100 mesh fines which was added as a substitute for cement was subtracted from this and the remainder represented the quantity of −100 mesh contained in the combined aggregate after mixing. All of Table 3 is in terms of aggregate, and does not include the −100 mesh sand which was substituted for cement.

Table 3 shows comparison of gradings, before and after mixing. Indicated changes in grading of the coarse aggregate, particularly cobbles, coarse rock and medium rock, were small and may have been due in part to the fact that samples for screen analyses were not strictly representative of the batch. For this reason no analysis of change in grading of the coarse aggregate has been attempted. Four of the six tests showed a small increase in total sand in the mix which is logical and was expected. In two of the tests, the paradoxical situation of an increase in plus 4-mesh material is indicated, but this is hardly possible and must be accounted for by the fact that samples used for determining "ingoing" combined aggregate grading were not representative of the batch.

The results of these tests altered somewhat the impression of the magnitude of some of the changes which had been concluded from the small "washdowns." The small tests had seemed to indicate that the minus 100-mesh almost doubled as a rule and sometimes even increased more than 100 per cent. The large tests indicated that on the average the minus 100-mesh increased about 60 to 70 per cent, but they substantiated the conclusion that the amount of grinding varied over a wide range. At least five conclusions were evident from the large washdown tests:

1. The minus 28-mesh material in the sand always increased, and the minus 4-mesh plus 28-mesh always decreased during mixing.

TABLE 3—COMPARISON OF AGGREGATE GRADING FOR MASS CONCRETE BEFORE ABD AFTER MIXING

Regular Charging

Test No.	**Percent Thru / On	Standard Screen Size 4 / 8	8 / 14	14 / 28	28 / 48	48 / 100	100 / Pan	FM Sand
6	Before Mixing	7.1	5.2	4.1	3.8	3.1	3.2	2.99
	After "	5.2	4.6	3.6	3.8	4.4	5.7	2.47
	*Percent Change	−26.8	−11.5	−12.2	0	+41.9	+78.1	
2	Before Mixing	5.9	4.9	3.8	3.4	3.6	3.6	2.81
	After "	5.5	4.3	3.5	4.8	4.1	5.3	2.50
	*Percent Change	− 6.8	−12.2	− 7.9	+41.2	+13.9	+47.2	
1	Befrre Mixing	6.5	4.8	3.9	2.9	4.2	4.3	2.76
	After "	5.2	4.2	3.6	3.7	6.8	6.8	2.24
	*Percent Change	−20.0	−12.5	− 7.7	+27.6	+61.9	+58.2	
		Split Charging						
3	Before Mixing	8.4	5.2	4.1	3.6	3.8	3.1	3.05
	After "	5.8	4.6	3.8	4.0	4.7	3.8	2.68
	*Percent Change	−31.0	−11.5	− 7.3	+11.1	+23.7	+22.6	
4	Before Mixing	6.4	4.8	3.8	4.1	3.2	3.2	2.90
	After "	4.5	4.3	3.4	4.9	4.0	4.0	2.53
	*Percent Change	−29.7	−10.4	−10.5	+19.5	+25.0	+25.0	
5	Before Mixing	6.2	4.9	3.9	3.7	3.5	3.1	2.89
	After "	4.5	4.3	3.4	4.1	3.7	5.2	2.46
	*Percent Change	−27.4	−12.2	−12.8	+10.8	+ 5.7	+67.7	

*Expressed as per cent change based on amount of material before mixing.
Minus indicates the quantity after mixing is less than it was before mixing.
Plus indicates the quantity after mixing is greater than it was before mixing..
**Expressed as per cent of total aggregate in the mix.

2. The plus 4-mesh minus ⅜ inch material usually decreased somewhat.

3. The plus ⅜ in. material changed very little.

4. The split batch generally showed less grinding than the regular batch.

5. The amount of change due to grinding varied widely for the individual sand fractions.

EFFECT ON UNIFORMITY OF CONCRETE PRODUCED

The simple fact that grinding takes place during the mixing process was not a new disclosure, although it is probably not seriously considered by most observers. The factor which complicated the production of concrete of uniform quality in this instance was not so much that grinding took place during mixing, but rather that grinding was not uniform.

The variable nature of the material made it necessary to design the mix for a condition which was average most of the time. When conditions arose which deviated too much from this average it became necessary to adjust by trial to maintain desired workability. The addition of cement and water at the designed water-cement ratio was permitted, but only as a last resort or as a temporary expedient of short duration until the unfavorable condition was corrected. The addition of cement to maintain required workability was avoided, first, because of the added tendency toward cracking due to the additional heat generated, and, second, because it is a very expensive procedure. The effect of the variable grinding on the production of concrete of uniform quality was magnified by the fact that very lean and very dry concrete was the order of procedure. The "mass" concrete which composed the interior of the dam contained only 0.80 bbl. of cement per cu. yd. and had a "wet screened" slump of less than one inch as a rule.

FACTORS WHICH INFLUENCE GRINDING

With all other conditions remaining constant, the physical characteristics of the aggregate will influence to a very great degree the amount and character of grinding which takes place during mixing. The structural composition of the aggregate has probably the greatest influence on amount and character of grinding; whereas, the particle shape, surface texture, maximum size, and grading, exert influences to a lesser degree in about the order named. With an aggregate of given physical characteristics, the amount and character of the grinding will vary with the characteristics of the mixer. For a given mixing time the size of the mixer and the arrangement of the blades will probably have a very great influence on the grinding. The mixing time will also influence the amount of grinding. It can be safely stated that grinding continues throughout the mixing period, but it does not necessarily follow that grinding is uniform throughout the mixing period. With the "graywacke" the grinding was much more rapid in the first half of the mixing period than in the last half. It was also demonstrated that the manner of charging influenced the amount of grinding in this particular instance, and it seems logical that this will be true with any material.

CONCLUSIONS

The experience at Hiwassee justifies several conclusions in regard to grinding in general:

1. Grinding takes place to some degree in all mixers.

2. Comparison of properties of concrete affected by grading should be made on the basis of grading of the aggregate in the mixed concrete, rather than the grading of the aggregate before mixing takes place.

3. In preparing grading specifications for aggregates, particularly sand, consideration should be given, if possible, to the effect of changes in grading due to grinding in the mixer.

4. When investigating qualities and properties of aggregates, the aggregate having the most uniform grinding characteristic and the one showing least amount of grinding, should be given preference, all other things being equal. Full appreciation of the value of this conclusion can be gained only by attempting to produce concrete of uniform quality from an aggregate of widely variable grinding qualities.

ACKNOWLEDGMENTS

Credit is due Ivan L. Tyler, formerly Senior Materials Engineer, Tennessee Valley Authority, (now Research Engineer, Portland Cement Association) for being first to realize that grinding of the graywacke in the large mixers would be a problem. Preliminary concrete investigations, including investigation of the grinding problem, were under direction of Mr. Tyler. Field investigations were started under direction of Mr. Tyler and continued under direction of the Author. The Author is indebted to Mr. Tyler for reviewing and criticizing the paper.

Theodore B. Parker is Chief Engineer, and A. L. Pauls is Chief Construction Engineer for the Tennessee Valley Authority. At Hiwassee Dam, C. E. Blee was Project Manager; O. Laurgaard was Construction Engineer; and, J. E. Walters was Construction Superintendent.

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Developments in Methods of Testing and Specifying Coarse Aggregates*

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Member American Concrete Institute

SYNOPSIS

In suggesting a different basis for testing the screening of coarse aggregate, emphasis and restriction are placed on that portion of the undersize material which is most detrimental to the production of uniform, high quality concrete. Clean separation on a production basis is impracticable; some material slightly smaller or slightly larger than the designated nominal size range of a coarse aggregate size fraction can be allowed without perceptible ill effect on the concrete. Characteristics of the undersize and common practice in the screening and handling of aggregates are discussed. A test for determination of only the "significant" (objectionable) portion of the nominal undersize is submitted, and a specification based on this test.

The common method of testing to determine whether coarse aggregate is acceptably screened to designated size fractions is by screen analyses for oversize and undersize, using screens having openings of the same dimensions as the nominal size limits of the aggregate fractions.

Although it has long been in use, this procedure does not constitute a realistic approach to the problem it purports to solve in that it makes no distinction between significant elements of undersize which, from a practical standpoint, can be reduced by good screening and elements of little or no importance which cannot. Because of this, wide tolerances are necessary and in these wide tolerances the elements which are significant are hopelessly scrambled with elements which are not. Until tests and procedures are developed and used which positively control significant variations in aggregate as batched, much that we call concrete control will continue to be largely futile.

By "significant," in relation to undersize, is meant the material so much smaller than the nominal size of the aggregate that it is practically

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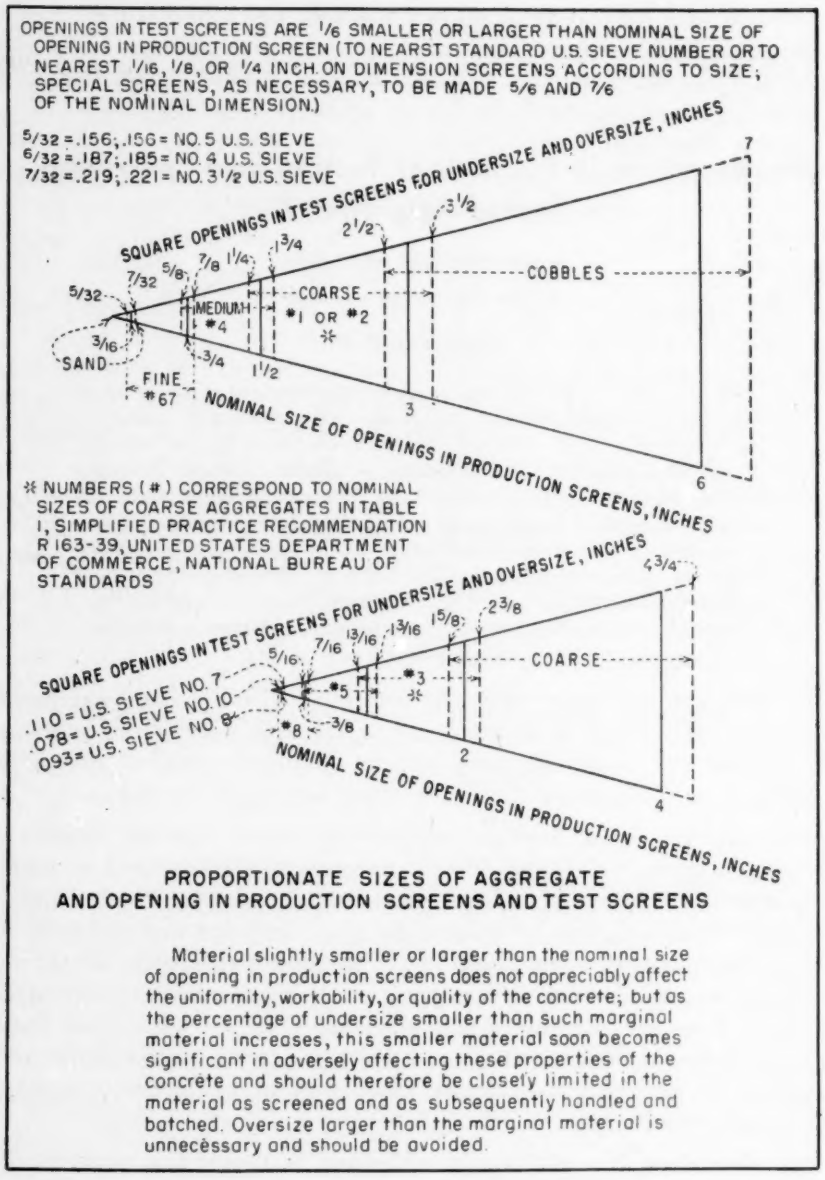
†Bureau of Reclamation, Denver.


OPENINGS IN TEST SCREENS ARE 1/6 SMALLER OR LARGER THAN NOMINAL SIZE OF OPENING IN PRODUCTION SCREEN (TO NEARST STANDARD U.S. SIEVE NUMBER OR TO NEAREST 1/16, 1/8, OR 1/4 INCH ON DIMENSION SCREENS ACCORDING TO SIZE; SPECIAL SCREENS, AS NECESSARY, TO BE MADE 5/6 AND 7/6 OF THE NOMINAL DIMENSION.)
5/32 = .156; .156 = NO. 5 U.S. SIEVE
6/32 = .187; .185 = NO. 4 U.S. SIEVE
7/32 = .219; .221 = NO. 3 1/2 U.S. SIEVE
SQUARE OPENINGS IN TEST SCREENS FOR UNDERSIZE AND OVERSIZE, INCHES
5/32
7/32
5/8
7/8
1 1/4
1 3/4
2 1/2
3 1/2
7
SAND
3/16
FINE #67
MEDIUM #4
COARSE #1 OR #2 *
COBBLES
3/4
1 1/2
3
6
NOMINAL SIZE OF OPENINGS IN PRODUCTION SCREENS, INCHES
* NUMBERS (#) CORRESPOND TO NOMINAL SIZES OF COARSE AGGREGATES IN TABLE 1, SIMPLIFIED PRACTICE RECOMMENDATION R 163-39, UNITED STATES DEPARTMENT OF COMMERCE, NATIONAL BUREAU OF STANDARDS
SQUARE OPENINGS IN TEST SCREENS FOR UNDERSIZE AND OVERSIZE, INCHES
4 3/4
2 3/8
1 5/8
1 3/16
13/16
7/16
5/16
.110 = U.S. SIEVE NO. 7
.078 = U.S. SIEVE NO. 10
.093 = U.S. SIEVE NO. 8
COARSE
#3 *
#5 *
#8 *
3/8
1
2
4
NOMINAL SIZE OF OPENINGS IN PRODUCTION SCREENS, INCHES

PROPORTIONATE SIZES OF AGGREGATE AND OPENING IN PRODUCTION SCREENS AND TEST SCREENS

Material slightly smaller or larger than the nominal size of opening in production screens does not appreciably affect the uniformity, workability, or quality of the concrete; but as the percentage of undersize smaller than such marginal material increases, this smaller material soon becomes significant in adversely affecting these properties of the concrete and should therefore be closely limited in the material as screened and as subsequently handled and batched. Oversize larger than the marginal material is unnecessary and should be avoided.

Fig. 1—Acceptable practical ranges of nominal aggregate sizes.

certain sooner or later to segregate and become so concentrated in portions of the supply that poorly graded batches, difficult to work, are at times inevitable. By "significant," in relation to oversize, is meant the material which indicates that a screen is overworn or broken and which, in constricted situations, may cause trouble in placing the concrete.

Some specifications permit up to 20 percent of nominal undersize with no mention of oversize, yet every hour a screen is in operation, the maximum size of the aggregate passing is increasing due to enlargement of the openings from wear. Simplified practice specifications permit up to 15 percent undersize and up to 10 percent oversize, total 25 percent, enough to make it appear, at least, that screening had been very crude. The implication of such large tolerances is that good screening is difficult to obtain economically and that considerably less than good screening is good enough for the great majority of concrete work. Actually neither is true. Good screening, as later described, is practicable on the basis of control tests concerned only with significant departures from the designated nominal range in size.

Clean separation on a production basis is impracticable, but some material slightly smaller or slightly larger than the designated nominal size range of a coarse aggregate size fraction can be allowed without perceptibly affecting the concrete quality. The proposed size limits of such acceptable marginal material, from practical considerations, are 5/6 of the nominal minimum dimension for undersize and 7/6 of the nominal maximum dimension for oversize, both to the nearest standard United States sieve number or to the nearest 1/16-, 1/8-, or 1/4-in. on dimension screens according to size. However, when special screens are made up for these tests, the openings should be made as close to 5/6 and 7/6 of the nominal size as is practicable. These limits are shown graphically in Fig. 1 for a number of common sizes of coarse aggregate, including several sizes from the Simplified Practice recommendation of the Department of Commerce.

Material within the 1/6-size margins of nominal undersize and oversize will undoubtedly occur always in any aggregate screened and handled on a production basis. But, as pointed out in the note in Fig. 1, material within the proposed 1/6 limits exerts no harmful effect. It may therefore be ignored in determining the percentage of significant undersize and oversize. The increased size range due to the 1/6 margins is not sufficient to cause an appreciable increase in segregation and the average aggregate size, for practical purposes, would remain unchanged for computations of particle interference, size distribution, or surface area.

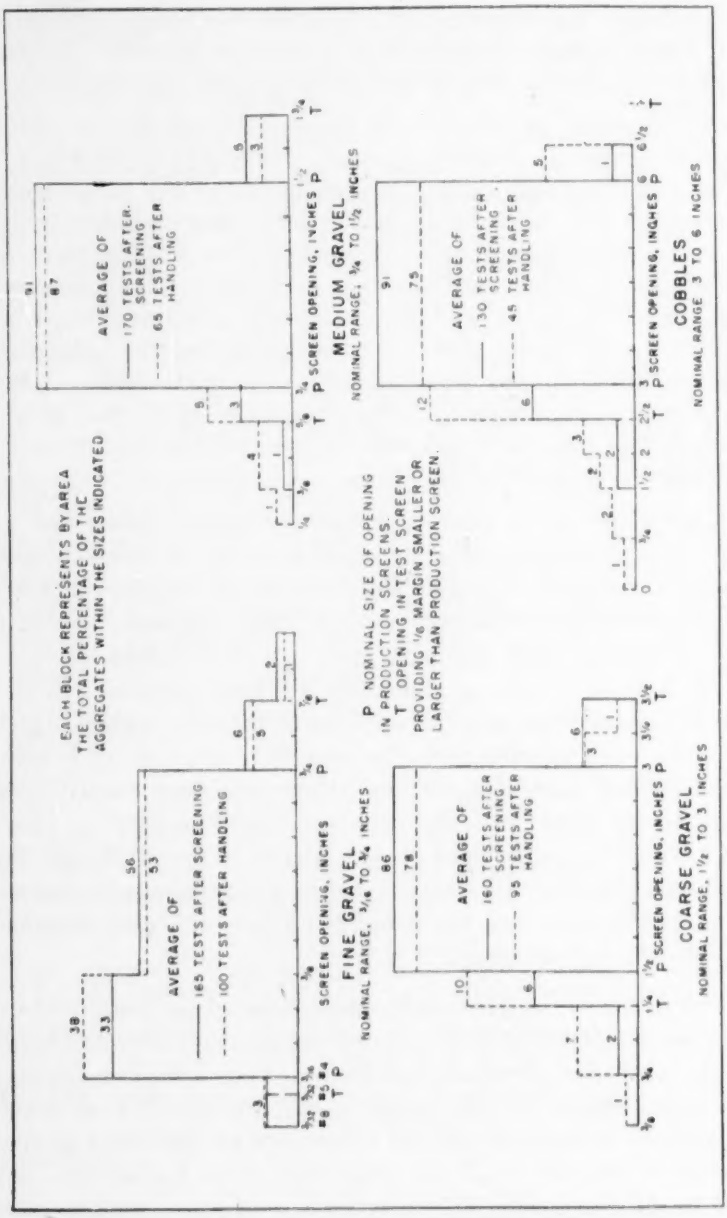


Fig. 2—Screen analyses of coarse aggregates, Shasta Dam

In connection with oversize, the 1/6 marginal allowance for all except the smallest sizes permits appreciable screen wear, the amount varying, of course, according to the weight of wire used. This is shown in Table 1.

TABLE 1—SCREEN WIRE SIZES FOR SQUARE OPENINGS

Size Opening, Inches	1/6 Margin, Inches	Screen Wire Sizes for Tyler Standard Clear Openings, Inches			
		Light	Standard	Medium	Heavy
(3/16) 0.187	0.031	0.080	0.092	0.120	0.135
(3/4) 0.75	0.125	0.192	0.207	0.250	0.312
1½	¼	¼	5/16	3/8	7/16
3	½	7/16	½	5/8	¾
6	1	5/8	¾	⅞	1 (Est)

The allowance of the 1/6 margin for oversize will not, as it may at first appear, encourage use of production screens with openings appreciably larger than the designated nominal size, because if larger openings are used, rejections for oversize above the margin would commence in most cases long before the screens were worn out. In the larger sizes where a little margin over a full allowance for wear exists, increases in production screen openings to this extent would be unimportant and give the operator a slight advantage in meeting undersize requirements in the next larger size.

Fig. 2 is a summary of several hundred screen analyses on the undersize and oversize in four sizes of aggregate produced during a 3-month period from natural deposits for use in Shasta Dam, under construction by the Bureau of Reclamation. The tests were made as part of a continuing study of plant performance by the Bureau looking toward the development of more effective, and yet at the same time more practicable specifications. The average data are presented in an area block chart for better emphasis and clarity. The areas for the various size fractions are proportional to the actual average amounts (numerals at the top of each block) of those sizes in the aggregate represented. The relation is shown between the portion of an aggregate corresponding to clean separation, the marginal material 1/6 smaller and larger, respectively, than the nominal minimum and maximum size, and the amounts and sizes of material in the significant undersize and oversize outside these margins.

It is evident from Fig. 2 that an undersize test screen with openings 1/6 smaller than those in the production screen will divide the nominal undersize in all sizes but fine gravel at such a point that the major por-

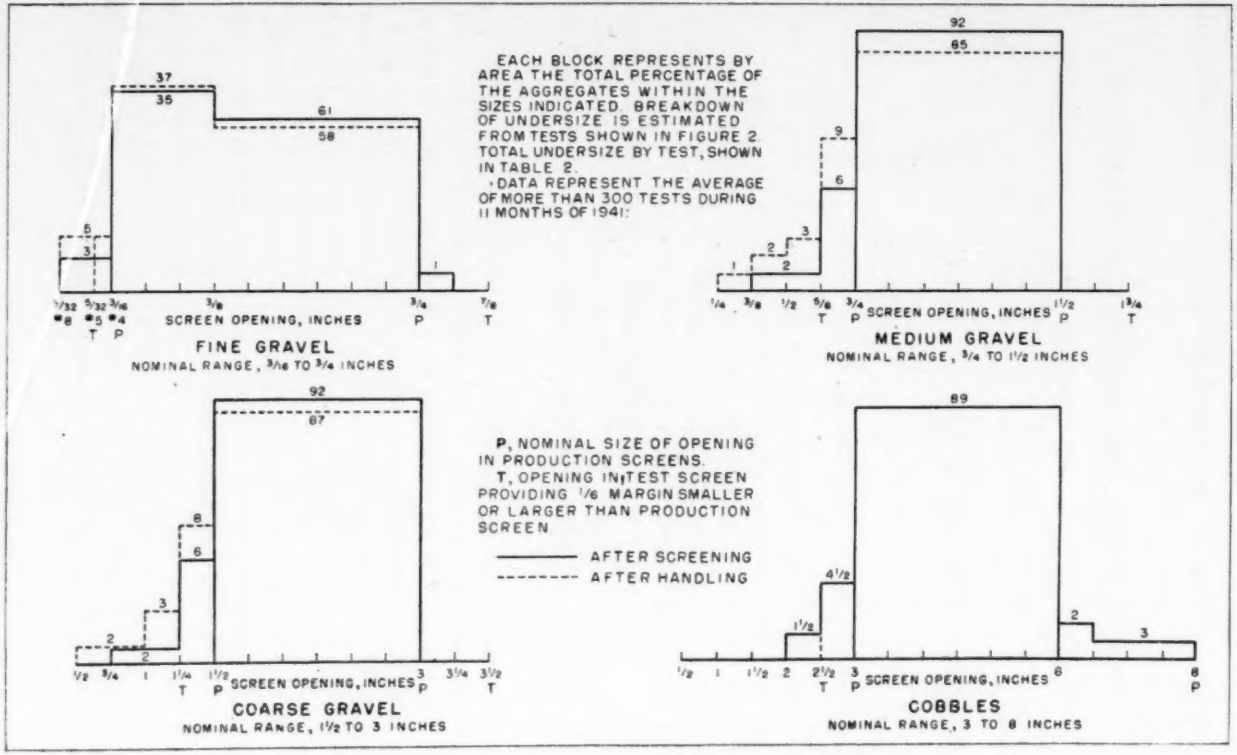

EACH BLOCK REPRESENTS BY AREA THE TOTAL PERCENTAGE OF THE AGGREGATES WITHIN THE SIZES INDICATED. BREAKDOWN OF UNDERSIZE IS ESTIMATED FROM TESTS SHOWN IN FIGURE 2. TOTAL UNDERSIZE BY TEST, SHOWN IN TABLE 2.
DATA REPRESENT THE AVERAGE OF MORE THAN 300 TESTS DURING 11 MONTHS OF 1941.
P, NOMINAL SIZE OF OPENING IN PRODUCTION SCREENS.
T, OPENING IN TEST SCREEN PROVIDING 1/6 MARGIN SMALLER OR LARGER THAN PRODUCTION SCREEN.
AFTER SCREENING
AFTER HANDLING
SCREEN OPENING, INCHES
FINE GRAVEL
NOMINAL RANGE, 3/16 TO 3/4 INCHES
MEDIUM GRAVEL
NOMINAL RANGE, 3/4 TO 1 1/2 INCHES
COARSE GRAVEL
NOMINAL RANGE, 1 1/2 TO 3 INCHES
COBBLES
NOMINAL RANGE, 3 TO 8 INCHES

Fig. 3—Screen analyses of coarse aggregates, Friant Dam.

tion of the nominal undersize is within the 1/6 smaller margin. Hence the choice of a margin corresponding to 1/6 of the screen opening is not entirely arbitrary but is based on consideration of many analyses of undersize.

Except in the fine gravel, 3/4 of the nominal undersize is within the 1/6 margin immediately after screening. In none of the aggregates, when first leaving the production screens, does the average undersize, smaller than the proposed 1/6 smaller test screen, exceed 2 percent of the material. That this is not an exceptionally low result, and one that is close to a limit that would define good screening, is indicated when comparison is made with the average of eleven months' screening performed during construction of the Friant Dam for the Bureau of Reclamation, as shown in Table 2 and Fig. 3. At the Friant plant, the average percentage of undersize was approximately the same (Table 2) as the average percentage of undersize shown in Fig 2, yet the screen openings in the Friant plant were such that practically no nominal oversize was produced and, of course, the series of aggregate fractions was continuous. Evidently the screening represented by Fig. 2, compared with that shown in Fig. 3, is considerably less than exceptional even though the average significant undersize does not exceed 2 percent of the material after screening.

There is comparatively little increase in undersize after handling the fine gravel, as shown in Fig. 2 and 3, probably due to much less impact among its much smaller particles. Although fine gravel is in itself the most critical of coarse aggregate fractions in influencing uniformity and workability* (because increased portions of fine aggregate or segregation within fine gravel place an increased demand on the margin of extra mortar provided to insure workability), trouble from undersize in fine gravel is usually much less than that caused by undersize in the larger aggregate fractions, for a number of reasons. First, the actual quantity of undersize from the fine gravel in a mix is less because in a good grading there is generally less of fine gravel than of each of the other coarse aggregates and because there is usually less undersize in the fine gravel than

*In the fine gravel sizes particularly, narrower than usual size ranges would contribute appreciable improvement to the uniformity and control of workability in concrete. For example, in No. 4 to ¾-in. material, the ratio of the maximum to minimum nominal size is 4. Disproportionate amounts of No. 4 to ⅜-in. and ⅜- to ¾-in sizes *within* the nominal limits of the material, due either to lack of balance in the raw material or to segregation later, are often responsible for more difficulty with the mix than are normal quantities of undersize from all sources. Although an ideal solution of this problem would be another division of the aggregates, in this case at the ⅜-in. size, probably the most readily acceptable solution is an adjustment of all the coarse aggregate sizes so as to reduce materially the spread in the ratios of the maximum to minimum nominal size in each fraction. The current divisions at No. 4, ¾, 1½, 3, and 6-in. sizes produce four aggregates having size ratios of 4, 2, 2, and 2. A more desirable division would be at the No. 4, ½, 1¼, 2¾, and 6-inch sizes because the ratios of maximum to minimum size in each fraction would then be 2.67, 2.50, 2.20, and 2.18. The increase in the ratios for the larger aggregates from 25 to 9 per cent would not be likely to produce perceptible disadvantages; whereas, the reduction of the ratio for fine gravel from 4 to 2.67 would result in visibly greater freedom from segregation and afford a means of controlling an unbalanced grading of materials.

TABLE 2—AVERAGE PERCENTAGE OF UNDERSIZE AND OVERSIZE IN COARSE AGGREGATE

Tested	Aggregate Fraction	Shasta Dam 3 Mo., 1940 (See Figure 2)	Friant Dam 11 Mo., 1941 (See Figure 3)
Fine Gravel: Nominal Size, $\frac{3}{16}$" to $\frac{3}{4}$"; No. 5 ($\frac{5}{32}$") Screen for Undersize Test			
After Screening	Passing $\frac{3}{16}$"	3	3
	Passing No. 5	2	2*
	Percentage of undersize significant	67	67*
	Oversize	8	1
After Handling	Passing $\frac{3}{16}$"	3	5
	Passing No. 5	2	3*
	Percentage of undersize significant	67	60*
	Oversize	6	None
Medium Gravel: Nominal Size, $\frac{3}{4}$" to 1$\frac{1}{2}$"; $\frac{5}{8}$" Screen for Undersize Test			
After Screening	Passing $\frac{3}{4}$"	4	8
	Passing $\frac{5}{8}$"	1	2*
	Percentage of undersize significant	25	25*
	Oversize	5	None
After Handling	Passing $\frac{3}{4}$"	10	15
	Passing $\frac{5}{8}$"	5	6*
	Percentage of undersize significant	50	40*
	Oversize	3	None
Coarse Gravel: Nominal Size, 1$\frac{1}{2}$" to 3"; 1$\frac{1}{4}$" Screen for Undersize Test			
After Screening	Passing 1$\frac{1}{2}$"	8	8
	Passing 1$\frac{1}{4}$"	2	2*
	Percentage of undersize significant	25	25*
	Oversize	6	None
After Handling	Passing 1$\frac{1}{2}$"	18	13
	Passing 1$\frac{1}{4}$"	8	5*
	Percentage of undersize significant	44	38*
	Oversize	4	None
Cobbles: Nominal Size, 3" to 6" (or 8"); 2$\frac{1}{2}$" Screen for Undersize Test			
After Screening	Passing 3"	8	6
	Passing 2$\frac{1}{2}$"	2	1$\frac{1}{2}$*
	Percentage of undersize significant	25	25*
	Oversize	1	None
After Handling	Passing 3"	20	No Tests
	Passing 2$\frac{1}{2}$"	8	" "
	Percentage of undersize significant	40	" "
	Oversize	5	" "

* Estimated from actual tests of undersize at Shasta Dam.

in the larger aggregates. Second, due to being smaller, fine gravel remains wet longer than larger material; as a result, there is less separation of undersize than in the larger material and a large source of trouble is minimized. Furthermore, the normal undersize in the fine gravel results only in an increase in the coarseness of the sand; whereas, undersize from the larger gravels increases the amount of fine gravel in the mix. The latter condition, as previously explained, causes a greater decrease in workability. In cases where the sand grading thus becomes too coarse, appropriate reductions in the percentage of fine gravel must be made.

Attention is particularly directed, however, in Fig. 2, to the difference in grading and amount of undersize after the screened material other than fine gravel has received considerable handling. The amount of significant undersize in the nominal undersize is almost doubled after storage and handling. Not only is the amount of significant undersize in the nominal undersize increased, but the percentage of this objectionable material in the aggregate as a whole is increased four or five times what it was as the aggregate left the screens. This is a point that should receive more than passing consideration for it is all too clear that the results of highly efficient screening are obliterated when the aggregates are subsequently handled in procedures that are all too common on the average job. For instance, as shown in Table 2, at Friant Dam where the plant screening appears to have been better than at Shasta Dam and where handling was different in many important respects, the percentages of nominal undersize in the aggregates as batched were in general as great as they were at Shasta Dam. Presumably the significant undersize was proportional but no specific tests are available. Based on these comparisons and other similar experiences, the following changes in common practice are recommended. It will be evident that some of these changes are not readily adaptable to work underway but that they will usually be found practicable for adoption for plants and jobs in the planning stage.

All stockpiling not absolutely necessary should be avoided. Aggregate should be kept off the ground. Whenever possible, batch trucks supplying mixers at the forms, truck mixers,* or agitator trucks hauling centrally mixed concrete, should be used between the aggregate plant and the forms so as to avoid all intermediate handling and stockpiling on noncentralized work. In such a set-up, an arrangement whereby well screened material passed directly to the batcher bins would very nearly eliminate variation in the concrete due to the changes in aggregate grading. Final screening at the batcher bins of a roughly proportioned blend of coarse aggregates from stockpiles of washed and roughly screened

*For a method of procedure for controlling consistency when using truck mixers, see ACI JOURNAL, Jan. 1942, p. 269. ("Transit Mixers Used Effectively on the Contra Costa Canal," by O. G. Bodin.)

material is particularly desirable for work involving large quantities of mass concrete.* The uniformity and workability of concrete of this class is much more sensitive to varying concentrations of undersize because mass concrete contains a smaller percentage of mortar and thus has a lower margin of extra mortar available to offset the effects of sudden increases in the content of small, usually harsh, undersize coarse aggregate. The benefits in undersize control to be gained by such a procedure are clearly indicated by the comparatively low percentage of undersize in aggregate that has not been subjected to handling as shown in Fig. 2.

No one standard form of specification for coarse aggregate can be written to cover all the situations which should be provided for on different jobs. However, the following paragraphs relative to undersize and oversize and methods of testing for the significant amounts of such outsize material, are suggested as a basis for future specifications for aggregate screening and grading. These considerations have received attention in the preparation of recent Bureau of Reclamation specifications for Davis Dam. The number and range of sizes, although desirable, are shown only to make complete text. Appropriate sizes and corresponding test screens can be readily substituted for particular conditions.

SUGGESTED SPECIFICATIONS

a)—Coarse aggregate shall range in size from a nominal minimum of $\frac{3}{16}$ in. to a nominal maximum of 6 in. Within this range the coarse aggregate shall be separated into four nominal-size fractions, as follows:

Designation of Fraction	Nominal Size Range
½ in. aggregate	$\frac{3}{16}$ in. to ½ in.
1¼ in. aggregate	½ in. to 1¼ in.
2¾ in. aggregate	1¼ in. to 2¾ in.
6 in. aggregate	2¾ in. to 6 in.

b)—Separation of the coarse aggregate into the four nominal-size fractions shall be such that when any fraction, as it leaves the screening plant, is tested on the screens hereinafter designated, the percentage of (significant) undersize, by weight, shall not exceed .....(2 − 3)† per cent, and there shall be no (significant) oversize.

*This suggestion is more fully discussed in Transactions of the ASCE, Vol. 106 (1941), p. 1272.

†Specification of a limit within the ranges thus indicated is believed practicable and considered desirable. The designated percentage will, of course, vary with special conditions.

Nominal Size Fraction	Size of Square Opening in Test Screen(1)	
	For Undersize	For Oversize
½ in. aggregate	$\frac{5}{32}$ in.(2)	$\frac{9}{16}$ in.
1¼ in. aggregate	$\frac{7}{16}$ in.	1½ in.
2¾ in. aggregate	1 in.	3¼ in.
6 in. aggregate	2¼ in.	7 in.

(1) Unless special screens can be obtained having openings more exactly 5/6 of the nominal minimum size for testing undersize or 7/6 of the nominal maximum size for testing oversize.
(2) No. 5 U. S. standard sieve.

c)—Handling of the aggregates after screening shall be such that the undersize as increased due to breakage and segregation, when determined by tests of samples of the aggregates as batched, and made in accordance with the method specified for determining the undersize in coarse aggregate as it leaves the screening plant, shall not exceed .....2 − 3)* per cent for the ½-in. aggregate, ...(3 − 4½) per cent for the 1¼-in. aggregate, ...(4 − 6) per cent for the 2¾-in. aggregate, and .....(5 − 7) per cent for the 6-in. aggregate. (Fig. 2 shows that the larger aggregates naturally develop a greater amount of significant undersize with handling; thus an increasing, graduated scale of limits is more appropriate than one limit for all sizes.)

d)—The engineer will test the aggregate and the contractor shall provide facilities that the engineer may require for ready procurement of representative samples. In the event that a routine test indicates nonconformity with specification requirements, two additional samples will be taken promptly and tested, and the average of the results of the three tests will be used in determining the acceptability of the aggregate.

Stipulation of individual limits for significant undersize in the newly screened material and in the material as batched has the advantage of bringing out sharply the two elements contributing to undersize in the batched material: (1) screening and (2) handling. This should be beneficial, both in the performance of the work and in the administration of the specifications by making both elements more objective and thus giving better assurance of obtaining the desired results. However, on work where only one operator will be responsible for both screening and subsequent handling, there may be conditions under which it may be advisable to limit the significant undersize and oversize at one point only; that is, as the material is batched. In this case, part (*c*) of the suggested specification would be omitted and part (b_1) would be substituted for part (*b*).

b_1)—Separation of the coarse aggregate into the four nominal-size fractions, and subsequent transportation, stock piling, and other handling operations, shall be such that when any fraction as batched is tested on the screens hereinafter designated, the percentage of (significant) undersize, by weight, shall not exceed .....(2 − 3) per cent for the ½-inch aggregate, ...(3 − 4½) per cent for the 1¼-inch aggregate, ...(4 − 6) per cent for the 2¾-inch aggregate, and .....(5 − 7) per cent for the 6-inch aggregate, and there shall be no (significant) oversize. (Table of designated

*See footnote (†) page 30.

screens same as in part *b*.) (An increasing, graduated scale of limits is necessary when the effects of handling are included.)

Neither of the above specifications makes it necessary for a contractor to perform the finish screening at the batcher bins. This can be made necessary by (1) lowering the limits on undersize at the batchers sufficiently to make a screening over the batcher bins practically mandatory (as was done to a large extent during construction of the Colorado River aqueduct) or (2) specifying it directly as follows (b_2) in another alternate for part (*b*) and omitting part (*c*).

b_2)—Finish or final screening of the coarse aggregate into the four nominal-size fractions shall be performed at or over the bins above the batchers in such a manner that when any fraction as batched is tested on the screens hereinafter designated, the percentage of (significant) undersize, by weight, shall not exceed .....(2 − 3) per cent, and there shall be no (significant) oversize. (Table of designated screens same as in part *b*.)

Discussion of this paper should reach the ACI Secretary in triplicate by March 1, 1943 for publication in the JOURNAL for June, 1943

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Discussion of a paper by Lewis H. Tuthill:

Developments in Methods of Testing and Specifying Coarse Aggregates*

By A. H. D. MARKWICK, GORDON L. WILLIAMS and AUTHOR

By A. H. D. MARKWICK†

The grading of screened aggregates, considered by Mr. Tuthill in relation to concreting material, has been studied at Harmondsworth in another connection. The shape of the typical size distribution histograms given in Fig. 2 and 3 of the paper arises from the screening process as shown in the explanation and idealized curve attached, and the existence of oversize and undersize is thus inherent in the commercial screening process as the author has indicated.

The uniformity of an aggregate, however, is as important as its grading and I should like to draw attention to two factors that the author has not mentioned. In commercial production, very appreciable differences in grading exist in the material that passes different parts of a screen because material that passes most easily passes first. There will be a tendency therefore, as the aggregate moves down any section of a screen with given apertures, for the average size of material passing to increase progressively. This has been confirmed experimentally and both oversize and undersize are correspondingly affected. The material in the hopper beneath the screen will therefore, differ in size according to its position relative to the screen feeding it. In addition, it has been shown that the loading of the screen at any time has an important effect on the grading of the material produced. There thus exist in the screening equipment itself two important sources of variability quite apart from any segregation and breakage that may substantially occur in storage and transit.

In specifying coarse aggregate, the author proposes to set limits for oversize and undersize at 7/6 of the nominal maximum size and 5/6

*ACI JOURNAL, Sept. 1942; *Proceedings* V. 39, p. 21.
†Road Research Laboratory, Harmondsworth, England.

of the nominal minimum size respectively, and goes so far as to suggest the use of special test sieves for this purpose. I do not consider that there is any need to introduce new, non-standard sieves as the size distribution in both oversize and undersize is controlled by the screening process. All that is necessary to exclude "significant" oversize (and undersize) is to limit the amounts in size fractions that can be defined by existing standard sieves and to make suitable adjustments in the quantity of the "significant" undersize permitted. For example in British Standard Specification No. 63:1939—"Sizes of Roadstone and Chippings," nominal 1½-in. material is defined as follows:

Tolerance on Oversize				Specified Size			Tolerance on Undersize			
Passing B.S. Sieve	%	Retained B.S. Sieve	% Max.	Passing B.S. Sieve	Retained B.S. Sieve	% Min.	Passing B.S. Sieve	% Max.	Passing B.S. Sieve	% Max.
2-in.	100	1½-in.	15	1½-in.	1-in.	75	¾-in.	5	7-mesh	2

Although this material is more tightly graded than is necessary in concrete aggregates the same principles can be applied.

The appended abstract from the published work mentioned* makes clear the mechanism of size distribution in screened aggregates.

EXPLANATION OF SIZE DISTRIBUTION IN SCREENED AGGREGATES

Commercial sizes of stone contain an indefinite number of different particle sizes, i.e., the particle size distribution is continuous. The histogram in Fig. A is from a sample of ½-in. granite that has been hand-sorted on square metal gauges differing in size by only 0.025 in. If it were possible to decrease the difference between adjacent gauges indefinitely, while at the same time increasing the sample size to correspond, a smooth curve would result, which would approximate closely to the idealized "normal distribution" curve which has been superimposed on the histogram.

A simple explanation can be given of this size distribution (Fig. B). Consider the separation of material supposed to contain equal proportions of all the particle sizes present by a screen of aperture size OA. In practice the separation will not be perfect, as some material smaller than OA will be retained, and the particle size distribution of the screened products may be supposed to be given by curve (1), where ordinates above the curve represent the size distribution of material retained, and those below the curve that of material passing. Similarly, the separation effected by a screen of aperture size OB may be represented by curve (2).

*Markwick, A. H. D. "The Grading and Shape of Commercial Sizes of Aggregates." *Soc. Chem. Ind. J.* 1942, *61* (6), 85-91.

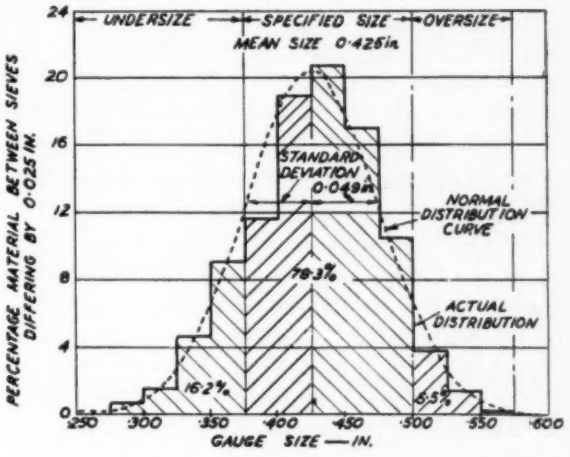

Fig. A—Distribution of size in a sample of 1/2-in. granite

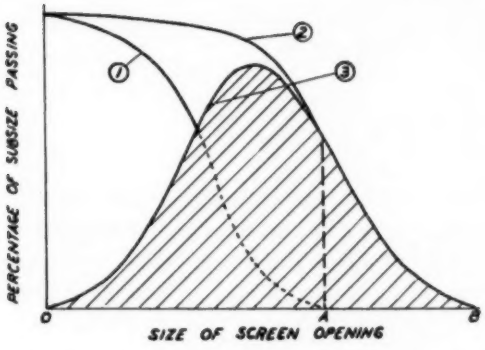

Fig. B—Explanation of size distribution found in commercial sizes of aggregates

1, Screen OA alone. 2, Screen OB alone. 3, Screen OA followed by screen OB (material retained on screen OA and passing screen OB shown hatched)

If the material retained by screen OA (i.e. material with the particle size distribution represented by the ordinates above curve (1)) be now passed over a screen having the larger opening OB, all the material larger than OB will be retained as before, but the proportion retained of the various subsizes smaller than OA will be less than that given by curve (2), since the proportion of the different particle sizes in the material fed on to the screen in this case varies from 100 per cent of size OA to zero (curve (1)). Thus the particle size distribution of the material between the two screens will be a combination of the screen characteristics given by curves (1) and (2), and will be of the form shown in curve (3), which is the type of distribution found (Fig. A). This explanation has been verified experimentally.

By GORDON L. WILLIAMS*

Having had a hand in collecting the Shasta Dam data presented in Mr. Tuthill's paper, it is gratifying to see it being put to practical use.

*Benton, Tennessee.

Ever since it was first mentioned to me by the aggregate contractor at Shasta Dam, I have been in favor of a slight overlapping of sizes in coarse aggregates coupled with a strict limitation of sizes which vary widely from the nominal. Obviously, these marginal sizes are the most expensive to remove, but do the least harm in concrete batching.

Mr. Tuthill's proposal to change the nominal sizes of the four coarse aggregates is an approach to the five sizes (3/8, 3/4, 1½, 3, and 6-in.) which many have advocated for some time. I believe most will agree that 3/16 to 3/4-in. is too wide a spread for one size of gravel. However, changing the sizes of test screens will do away with convenient fineness modulus information on coarse aggregates. Some may consider this objectionable.

If we change the nominal sizes of coarse aggregates into four somewhat equal sizes, why not make them nearer equal than the sizes Mr. Tuthill suggests? The conventional sizes listed above constitute a geometrical series with a ratio of two. If we go from 3/16 to 6 in. in four jumps instead of five, we can solve for the ratio r as follows:

$$\frac{3}{16} r^4 = 6$$

$$r = \left(\frac{6 \times 16}{3}\right)^{1/4} = 2.38$$

Our series then becomes 0.1875, 0.446, 1.06, 2.52 and 6. The common screen sizes nearest to these are: 3/16, 7/16, 1, 2½ and 6 in. This might be a slight improvement over Mr. Tuthill's proposed sizes of 3/16, ½, 1¼, 2¾, and 6-in. in providing a lower ratio in the smaller sizes where uniform grading is most essential.

AUTHOR'S CLOSURE

Mr. Williams has offered a good rule for equally spacing the size separation of coarse aggregate widely varying in size and such equal spacing, it is agreed, is the ideal. Sometimes considerations of the most practicable maximum size of aggregate for concrete not containing the larger of the sizes produced, may make it preferable to vary the intermediate sizes some fraction of an inch from the ideal. The advantages of retaining a given series of sizes in order to provide a perfect computation of the fineness modulus is considered inferior to the advantages of having a series of size ranges, which result in minimum segregation and from which most advantageous maximum sizes of aggregates for various placing conditions may be selected.

Mr. Markwick has correctly pointed out the importance of the influence of rate of feed and length of screen on the grading of the passing product. Unless it is assured that these are maintained at an efficient optimum, or at least constant, a series of variables is introduced which make most

discussion of this subject meaningless. In many cases the 5/6 and 7/6 nominal size test screens proposed correspond with one standard size or another although sometimes special test screens will be required. The point is that, according to the data investigated, screens of these sizes relative to the nominal, special or not, appeared necessary in order to designate consistently the significant undersize in each aggregate size group. When only nearest so-called standard screens are used to designate significant undersize, the significance (and with it the allowable percentage) of undersize so designated varies with the spread between the size of opening in the test screen and the nominal size, and in many cases the spread will be so great as not to measure the amount of all undersize that is significant. Although some may have reason to believe the dividing line between significant and unsignificant undersize and oversize is other than 5/6 and 7/6 of the nominal size, it is considered essential for consistent evaluations that the test screens always bear the same relation to the nominal size whether this results in requirement for a special test screen or not.

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The Design of Box Culverts*

By C. R. BURKY†

SYNOPSIS

In the Canal Engineering Section of the Bureau of Reclamation rectangular concrete box culverts are designed for a number of purposes. Box culverts are usually built to carry drainage water under canals, to carry drainage water under roads or railroads, or to carry canal water under roads, railroads, other canals, or natural drainage channels. In the work carried on by the Canal Engineering Section the most frequent use of box culverts is for the purpose of carrying drainage water under canals.

Some of the methods used in the design of box culverts are given herein, and while this is written from the standpoint of the design of box culverts, much of the procedure outlined would apply to any box structure.

INLET AND OUTLET

Fig. 1 outlines a typical drainage culvert under an earth canal. In general the inlet and outlet transition walls will be vertical or warped according to the relation of the culvert to the ground line at the ends of the culvert. Warped transitions are preferred as being somewhat cheaper and probably somewhat better from a hydraulic standpoint. However, it is not advisable to use a warped transition where the inlet or outlet is not practically all below the natural ground line.

Where a culvert is used to carry canal water under another structure or under a natural drainage channel, the inlet and outlet will generally be warped to fit the canal section, using a straight line transition where the saving of energy head is not important and using a smoothly curved transition where loss of head in the canal must be kept low.

The upstream end of the inlet floor of a drainage culvert should not be much higher than the downstream end. If there is too much difference in elevation much of the benefit of the upstream cut-off wall is lost. This difference should not be more than 1.5 ft. for 2 x 2 ft. culverts and not more than about 3 ft. for 6 x 6 ft. culverts. If the upstream end of the inlet floor is too high it may serve as a control for the drainage water

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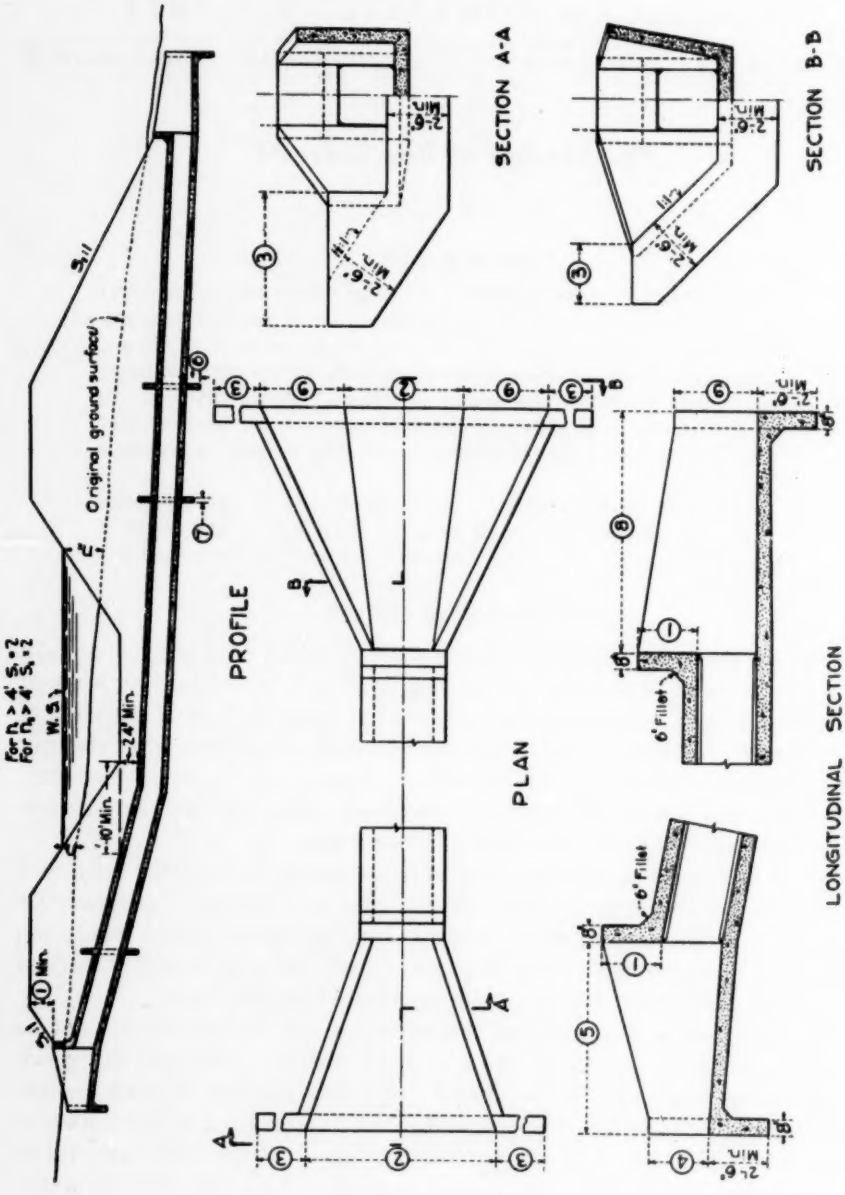

SECTION A-A
SECTION B-B
PROFILE
PLAN
LONGITUDINAL SECTION
Original ground surface
For $n_1 > 4'$ S:2
For $n_2 > 4'$ S:2
W.S.
24" Min.
10" Min.
6" Fillet
2'-6" Min.

Fig. 1

and hold the water level at this point at a higher elevation than would be required for entrance losses and velocity head at the entrance to the box. A set of dimensions has tentatively been set for drainage culverts commonly used, varying from 2 x 2 to 6 x 6 ft. (Table 1). This results in the same dimensions for inlets and outlets for the same size box and when bar placing diagrams, and bar bending lists have been prepared for a certain size box culvert they may be used, as to inlet and outlet transition, for any other culvert of that size. The use of the same data for bar placing diagrams and bar bending lists would not be possible if too much difference in elevation were allowed at the ends of a transition.

TABLE 1—TENTATIVE DIMENSIONS FOR DRAINAGE CULVERTS (FIG. 1)

Vertical or warped transitions may be used on either the inlet or outlet. In the table below "V" indicates vertical walls on transition, "W" indicates a warped transition.

Numbers 2, 3, 4 and 5 apply to inlet only; Numbers 8 and 9 apply to outlet only.

Size	Type	1	2	3	4	5	6	7	8	9
2x2 ft.	V	24″	6′-0″	5′-0″	24″	6′-0″	18″	6″	8′-0″	3′-0″
	W	24″	4′-0″	2′-6″	24″	6′-0″			8′-0″	3′-0″
3x3 ft.	V	2′-8″	9′-4″	6′-0″	3′-0″	9′-0″	24″	6″	12′-0″	4′-0″
	W	2′-8″	5′-4″	2′-6″	3′-0″	9′-0″			12′-0″	4′-0″
4x4 ft.	V	3′-0″	12′-0″	6′-0″	3′-0″	12′-0″	2′-6″	8″	16′-0″	5′-0″
	W	3′-0″	6′-0″	2′-6″	3′-0″	12′-0″			16′-0″	5′-0″
5x5 ft.	V	3′-0″	15′-0″	6′-9″	3′-6″	15′-0″	2′-6″	8″	20′-0″	6′-0″
	W	3′-0″	9′-0″	2′-6″	3′-6″	15′-0″			20′-0″	6′-0″
6x6 ft.	V	3′-0″	18′-0″	7′-6″	4′-0″	18′-0″	2′-6″	8″	24′-0″	7′-0″
	W	3′-0″	10′-0″	2′-6″	4′-0″	18′-0″			24′-0″	7′-0″
2 bbl. 4x4 ft.	V	3′-0″	20′-0″	6′-0″	3′-0″	12′-0″	2′-6″	8″	16′-0″	5′-0″
	W	3′-0″	16′-0″	2′-6″	3′-0″	12′-0″			16′-0″	5′-0″
2 bbl. 5x5 ft.	V	3′-0″	24′-0″	6′-9″	3′-6″	15′-0″	2′-6″	8″	20′-0″	6′-0″
	W	3′-0″	20′-0″	2′-6″	3′-6″	15′-0″			20′-0″	6′-0″
2 bbl. 6x6 ft.	V	3′-0″	34′-0″	7′-6″	4′-0″	18′-0″	2′-6″	8″	24′-0″	7′-0″
	W	3′-0″	25′-0″	2′-6″	4′-0″	18′-0″			24′-0″	7′-0″

The above dimensions are given as representative, from a list of 28 sizes of box drainage culverts used tentatively in design.

In the design of a vertical wall type of transition, the question arises as to the proper distribution of the earth load back of the wall.

In 1936, an analysis was made for vertical walls of uniform moment of inertia, by S. T. Larsen, then Junior Engineer, Bureau of Reclamation. The load was divided into triangles numbered 1 to 8 as shown in Fig. 2(c). Fig. 3 illustrates a vertical wall of the type considered. The wall was assumed as fixed at bottom and ends, and loaded with an earth load equivalent to a fluid pressure of 30 lb. per cu. ft. from top of wall. The loadings at the interior quarter points were then adjusted between

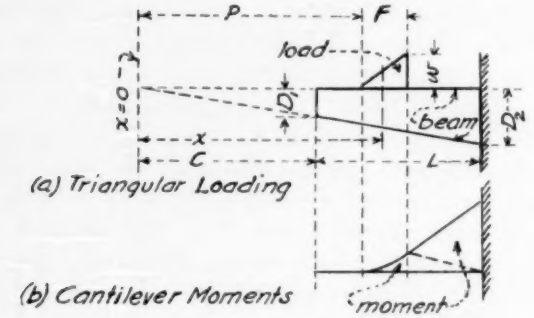
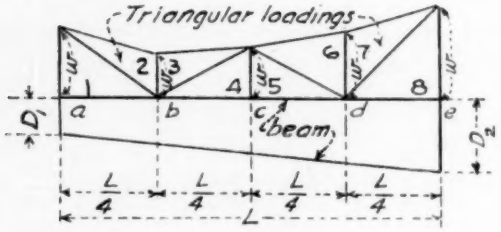


Fig. 2



(c) Triangular Loadings, lengths = $\frac{L}{4}$

vertical and longitudinal beam elements one foot wide, so as to make the deflections of the elements approximately the same at their intersections. Charts were prepared showing for any average height the moments at the ends of these elements and formulas were written to give an approximation of the moments at the interior quarter points, in terms of the final end moments.

To investigate the distribution of loads for beam elements having different thicknesses at the ends, a set of values for deflections of straight haunched beams for triangles of load was computed by I. B. Hosig, Engineer, Bureau of Reclamation. The method used is outlined below.

In addition to the nomenclature shown on Fig. 2, let D_x, I_x, M_x = depth, moment, and moment of inertia at any point x; I = moment of inertia at small (left) end; M or w = maximum intensity of moment or load in a triangle; E = modulus of elasticity, considered constant; $\overline{x}$ = distance from $x = O$, to centroid of $\frac{M_x}{EI_x}$ area. Then for a beam of uniform width, let $\frac{I}{I_x} = \frac{D_l^3}{D_x^3}$ and $\frac{D_l}{C} = \frac{D_x}{x}$, then $\frac{M_x}{EI_x} = \frac{M_x C^3}{EIx^3}$.

For a moment having the shape of triangle 6, Figure 2(c)

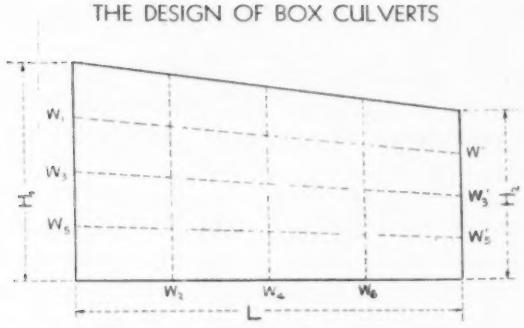


Fig. 3

Eq. 1 $\dfrac{M_x}{EI_x} = \dfrac{M}{F}\dfrac{(x-P)}{EI}\dfrac{C^3}{x^3}$ for any point x from $x = P$ to $x = P + F$

Eq. 2 $\displaystyle\int_{x=P}^{x=P+F} \frac{M_x}{EI_x} = \frac{MC^3F}{EI}\left[\frac{1}{2(P+F)^2P}\right] = \frac{M_x}{EI_x}$ area from $x = P$ to $x = P + F$

Eq. 3 $\displaystyle\int_{x=P}^{x=P+F} \left[\frac{M_x}{EI_x}(x)\right] = \frac{MC^3}{FEI}\left[log_e\frac{P+F}{P} - \frac{F}{P+F}\right] = \frac{M_x}{EI_x}$ area as above times $\bar{x}$.

For a load having the shape of triangle 6, Fig. 2(c)

Eq. 4 $\dfrac{M_x}{EI_x} = \dfrac{w}{6F}\dfrac{(x-P)^3}{EI}\dfrac{C^3}{x^3}$ for cantilever moment of load to left of any point x along the triangle of load.

Equation 4 can be integrated in a manner similar to equations 2 and 3. In like manner equations can be set up for load and moment similar to triangle 7 on Fig. 2(c). The conditions can be varied by changing the values of C, P, and F.

Fig. 2(a) shows a triangle of load on a haunched cantilever beam fixed at the large (right) and, with $x = 0$ located at a point such that the depth of the beam varies directly with x.

In Fig. 2(b) there is shown a diagram of the cantilever moment for the triangle of load shown in Fig. 2(a). This moment is defined by a curved line from $x = P$ to $x = P + F$ and can be considered as two triangles of moment from $x = P + F$ to $x = C + L$.

To compute the moment for a simple beam, the simple beam left reaction can be placed at the left end and the moment caused by this reaction can be added algebraically to the cantilever moments.

If the $\dfrac{M_x}{EI_x}$ area and its centroid can be computed for two triangles of moment, one with M at the right end and one with M at the left end,

and also for two triangles of load on a cantilever beam, one with w at the right end and one with w at the left end, the necessary data are at hand for computing deflections of cantilever, simple, and fixed beams, at points b, c, and d, Fig. 2(c).

A complete discussion of the principles involved in the use of moment areas was given by S. M. Cotten in his discussion on the Method of Conjugate Points[1], and these principles are also given in some text books.

Considering vertical walled transitions of dimensions given in Table 1, having bottom and ends fixed, the best method of arriving at design moments is in terms of the gross moment. These gross moments are computed assuming that the longitudinal beam elements carry all the load, and also that the vertical beam elements carry all the load, along these elements one foot wide.

For constant I, the maximum longitudinal moment will be at W_1 (Fig. 3), and will range from about 70 per cent of the gross moment for long transitions to 100 per cent of the gross moment for short transitions. For constant I, the maximum vertical moment will be at W_4 and this moment will range from about 80 per cent of the gross moment for short transitions, to 100 per cent of the gross moment for long transitions.

Increasing the thickness at the bottom of these walls in order to carry ordinary light earth loads has little effect on the load distribution. For a heavier haunching, say to resist a water load, the maximum longitudmoment will be smaller than for constant I and may be at W_3, while the maximum vertical moment at W_4 will be larger than for constant I. In one case for a short transition W_4 was nearly 20 per cent larger than for constant I.

TRANSVERSE MOMENTS IN BOX

The transverse moments in the box can be arrived at in a number of ways. This was once done by cumbersome formulas or arbitrary assumptions. With the publication of a paper by Hardy Cross[2] on the distribution of end moments, the determination of moments in a monolithic box was simplified. There is usually no loading on a culvert box which is unsymmetrical with respect to the vertical centerline and Professor Cross' assumption of no deflection of the joints with respect to each other will hold.

Referring to Fig. 4, assuming top and bottom of same thickness, and the sides of some other thickness, each member of uniform moment of inertia (I), all loads symmetrical with respect to the vertical center line of the structure, no deflection of joints, modulus of elasticity (E), the same for all members, a simple formula may be worked out to arrive at the final end moments directly. Let:

(1)See References, end of paper.

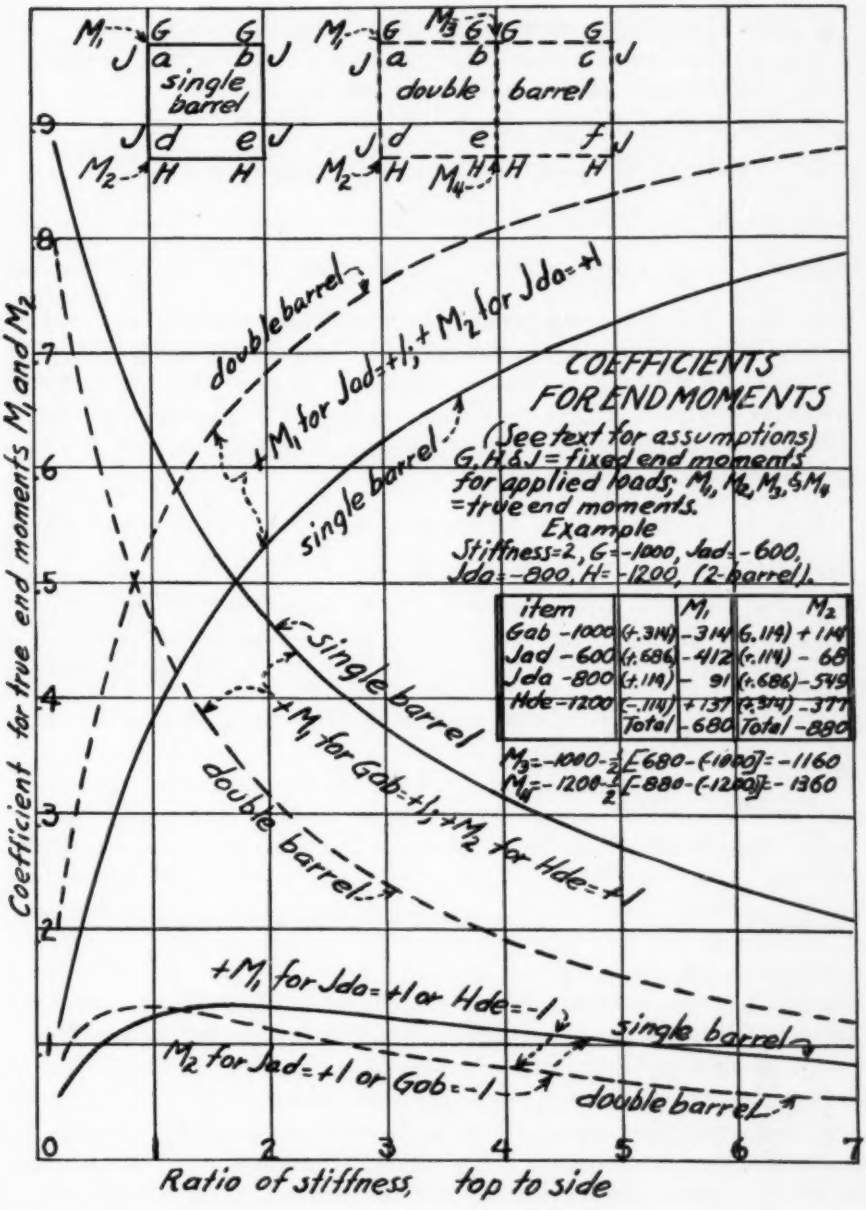

single barrel
double barrel
COEFFICIENTS FOR END MOMENTS
(See text for assumptions)
G, H & J = fixed end moments for applied loads; M1, M2, M3, & M4 = true end moments.
Example
Stiffness=2, G=-1000, Jad=-600, Jda=-800, H=-1200, (2-barrel).
item | M1 | M2
Gab -1000 (+.314) -314 (G.119) +119
Jad -600 (+.686) -412 (+.114) -68
Jda -800 (+.114) -91 (+.686) -549
Hde=1200 (-.114) +137 (+.314) -377
Total -680 Total -880
M3 = -1000 - 1/2 [-680 - (-1000)] = -1160
M4 = -1200 - 1/2 [-880 - (-1200)] = -1360
double barrel
+M1 for Jad=+1; +M2 for Jda=+1
single barrel
+M1 for Gab=+1; +M2 for Hde=+1
double barrel
+M1 for Jda=+1 or Hde=-1
single barrel
M2 for Jad=+1 or Gab=-1
double barrel
Coefficient for true end moments M1 and M2
Ratio of stiffness, top to side
0 1 2 3 4 5 6 7
1 2 3 4 5 6 7 8 9

Fig. 4

G, and H be fixed-end moments for the loads applied to the top and bottom respectively. (G *ab* indicates fixed-end moment at upper left hand corner due to top load, etc.)

$J\,ad$ = Fixed-end moment at top end of sides for load on sides.
$J\,da$ = Fixed-end moment at bottom end of sides for load on sides.
M_1 = Final end moment at upper left hand corner.
M_2 = Final end moment at lower left hand corner.
S = Ratio of stiffness of top to side (stiffness defined as $\frac{4EI)}{L}$.

By the use of fixed-end moments the structure will have an equal amount of positive and negative $\frac{M}{EI}$ area around the structure from any joint, the end slopes of all beams will be vertical or horizontal, and usually there will be a different end moment in the beams meeting at a joint.

Changes in the fixed-end moments must be made so that the final end moments will be equal on both sides of each corner. These changes must be made in a manner that will give the same slope at the ends of the two beams meeting at each corner.

Then for a single barrel taking reactions of the $\frac{M}{EI}$ areas of these changes from b to a equal to minus those from d to a (minus because of change of direction), and likewise from a to d equal to minus those from e to d, gives two equations for solving for the two unknowns M_1 and M_2. Moments causing tension on the outside of the box have the same sign. (Reactions of $\frac{M}{EI}$ areas indicate the slope of the beam at that end, in this case.)

For single barrel

Eq. 5—$(M_1 - Gab)\frac{3}{6} + (M_1 - Jad)\frac{2S}{6} + (M_2 - Jda)\frac{S}{6} = 0$

Eq. 6—$(M_1 - Jad)\frac{S}{6} + (M_2 - Jda)\frac{2S}{6} + (M_2 - Hde)\frac{3}{6} = 0$

From equations 5 and 6:

Eq. 7—$M_1(3+4S+S^2) = Gab(3+2S) + Jad(2S+S^2) + Jda(S) - Hde(S)$

Eq. 8—$M_2(3+4S+S^2) = -Gab(S) + Jad(S) + Jda(2S+S^2) + Hde(3+2S)$

For a double barrel box, using the same assumptions as to loading, etc., and considering top and bottom fixed at the vertical center line of the structure, any change at b or e will equal minus one-half the change at a or d, respectively. Equations may be written similar to those for a single barrel. For a double barrel

Eq. 9— $-\frac{1}{12}(M_1 - Gab) + \frac{1}{3}(M_1 - Gab) + \frac{S}{6}(M_2 - Jda) + \frac{S}{3}(M_1 - Jad) = 0$

Eq. 10—$\frac{S}{6}(M_1-Jad) + \frac{S}{3}(M_2-Jda) - \frac{1}{12}(M_2-Hde) + \frac{1}{3}(M_2-Hde) = 0$

From equations 9 and 10

Eq. 11—$M_1(3+8S+4S^2) = Gab(3+4S) + Jda(2S) + Jad(4S+4S^2) - Hde(2S)$

Eq. 12—$M_2(3+8S+4S^2) = -Gab(2S) + Jda(4S+4S^2) + Jad(2S) + Hde(3+4S)$

Eq. 13—M_3(Moment in top slab at center of structure $= Gba - \frac{1}{2}(M_1-Gab)$

Eq. 14—M_4(Moment in bottom slab at center of structure) $= Hed - \frac{1}{2}(M_2-Hde)$

A table of values was computed from these formulas, and curves were constructed from these values, in order to estimate the effect of a change of stiffness of the top relative to the side, for any particular end moment. These curves are shown on Fig. 4. For more than 2 barrels, or where loading or box is not symmetrical about the vertical center line, it is better to use the Hardy Cross method of moment distribution than to attempt to set up formulas. From Fig. 4, the effect of each of the four fixed-end moments on M_1 and M_2 can be ascertained.

It is seen that *Hde* has a negative effect on M_1 and that *Gab* has a negative effect on M_2. All other fixed-end moments have a positive effect. It is seen that a fixed-end moment at the top corner can never have a greater effect on M_2 than plus or minus 0.134 for any relative stiffness, and of course the same is true in regard to the effect on M_1 for any fixed-end moment at the bottom corner. For this reason these formulas would be fairly accurate even if the bottom slab was somewhat thicker than the top slab, if the stiffness ratio of bottom to side were used for *Hde* and *Jda*, and the stiffness ratio of top to side were used for *Gab* and *Jad*.

Curves shown on Fig. 4 were useful in arriving at end moments when each size of box was first designed. The design moments, however, are near the centers of beams for net "positive" moments and at the inside faces of connecting members for net "negative" moments. Span lengths are taken from center to center of connecting members.

A set of curves was drawn on one sheet for simple beam moments in terms of *WL* for uniform load, triangular load, and three trapezoidal loads having end load ratios of 2, 3, and 6 respectively. These curves were

plotted to a scale of 1 in. = 0.1 L horizontally and 1 in. = 0.02 WL vertically. The curves all lie fairly close together and the moment for any trapezoidal load can be interpolated by inspection. The final end moments could then be reduced to values in terms of WL for any beam and from a straight line through these points the net moment could be scaled in terms of WL for any point along the beam. The points of inflection could be located in terms of L on the horizontal scale. This method requires no plotting of moments, and no formula for points of inflection or maximum moment.

For design purposes the moment at some particular point in a certain box can be expressed by two constants and two straight line variables. For top loads due to fill there is a constant moment due to the box with no fill, and an additional amount of moment for each foot of fill. For any adopted fluid side pressure there is a moment due to the side pressure load up to the top of the box and an additional amount of moment for each added foot of side pressure height. In this manner the moment desired at this particular point can be read from a straight line chart for any fill and from another straight line for any height of side pressure independently. Moments due to fills are usually plotted on a basis of 100 lb. per cu. ft. Moments due to side loads are usually plotted for a fluid side pressure of 30 lb. per cu. ft. and also for a fluid pressure of 62.5 lb. per cu. ft.

For one or two barrels, where different side pressures may be under consideration, maximum "negative" moments in the sides will be obtained by using the maximum side pressure; maximum "positive" moments at the center of the bottom slab will be obtained by using the minimum side pressure; and for two barrels, the maximum "negative" moment in the bottom at the face of the center wall will be obtained by using the minimum side pressure.

For each particular box that has been designed, the designer keeps a chart or simple formula from which the design moments may be obtained directly for any condition of fill or side pressure.

For the convenience of the designers a tentative loading chart is used for the effects of trucks, etc., as given in Table 2.

For $h<2$ and for $h>2$ & $(h+1)<b$, no load on sides due to wheels.

For $h>2$ & $(h+1)\geqq b$, use $0.3\ P_{\tau}^{\square\prime}$, at top of side

$$\text{use } (0.3\ P_{\tau}^{\square\prime})\frac{(h+1-b)\ (1+I_B)}{(h+1-b+d)\ (1+I_\tau)}, \text{ at bottom of side}$$

$$\text{For truck approaching use } \frac{.6W(1+I_\tau)}{(w+6+h)\ (h+1)} \text{ at top of side } (P_{s}^{\square\prime})$$

$$\text{use } P_{s}^{\square\prime}\frac{(h+1)\quad (1+I_B)}{(h+1+\tfrac{1}{2}d)\ (1+I_\tau)}, \text{ at bottom of side}$$

For steam railroad, let Wa = load on one axle (70,000 where not given).

For $h<2$, $I = 1.00$; for $h>2$, $I = 1.00 - \frac{h-2}{11}$

$P_{\tau}^{\square'}$ (For wheels and track on top slab) $= \left[\frac{Wa(1+I\tau)}{5} + 200\right] \times \frac{1}{8+h}$

$P_{B}^{\square'}$ (For foundation reaction of wheels and track) $= \left[\frac{Wa(1+I_B)}{5} + 200\right] \times \frac{1}{8+h}$

For side loads due to wheels and track, use 0.3 $P_{\tau}^{\square'}$ at top of side and 0.3 $P_{B}^{\square'}$ at bottom of side.

For light electric lines use Wa = 53,000.

2½ cu. yd. drag line		Puddled Earth	
Fill above box	Pounds per sq. ft.	Height of side wall	Fluid Pressure pounds per sq. ft.
2	1250	1	100
3	1000	4	100
5	670	5	90
7	480	6	80
10	340	7	70
20	170	8	62.5

Earth is usually taken at 100 lb. per cu. ft. vertically and 30 lb. per cu. ft. horizontally. For sand requiring 2 to 1 slopes, use 40 lb. horizontally.

The box under the canal prism should be tested for the normal depth of water and no saturation; also with water at the normal depth with full hydrostatic pressure on the sides. An imaginary saturation line should be drawn from the intersection of the water surface with the bank, outward and downward on a slope of 4 horizontal to one vertical. The box under the banks should be tested for full hydrostatic pressure below this line, as well as for the regular earth side pressure. For some sandy soils the imaginary saturation line should be on a ratio greater than 4 to 1.

SIDE SWAY IN BOX

Side sway is seldom an important item in the design of a drainage culvert. There is usually a fill of several feet over the culvert. Side sway may have to be considered in the case of a highway culvert under practically no fill, or in the case of a square box siphon under a drainage channel, where the earth may be washed away on the downstream side.

Side sway presents the same sort of problem as the one where it is desired to compute a foundation reaction having a certain relation to the deflection of the bottom of a structure. In the consideration of side sway there is present an unsymmetrical side load resisted partly by the top of the box as a horizontal beam, partly by resistance of the sides of the box

TABLE 2

Weight in pounds, dimensions in feet.

b = gross width of culvert.
Subscripts B, T, and S indicate bottom, top and side.
d = gross height of culvert.
e and E = effective width of slab for span considered, or width of slab over which one wheel load is considered to be distributed.
h = fill above top of culvert.
I = impact expressed as a portion of the static live load. I is taken as 0.3 for fills 0 to 2 feet, and decreasing to zero at 5 feet. I_T is based on a fill of h feet; I_B is based on a fill of $(h + d)$.
P = pounds as a concentrated load on a beam one foot wide.
$P^{\square\prime}$ = pounds per square foot.
w = width of truck tire.
W = weight of one rear truck wheel.

	Loads due to Trucks			
	Top Slab		Bottom Slab	
Fill	1 Truck	2 Trucks	1 Truck	2 Trucks
$h<2$	$P_T = \frac{W(1+I_T)}{e}$	$P_T = \frac{W(1+I_T)}{E}$	$P_B^{\square\prime} = \frac{2W(1+I_B)}{(w+6+e)b}$	$P_B^{\square\prime} = \frac{4W(1+I_B)}{(w+15+e)b}$
$2<h<(6-w)$	$P_T^{\square\prime} = \frac{W(1+I_T)^*}{(w+h)\ (h+1)}$	$P_T^{\square\prime} = \frac{2W(1+I_T)^*}{(w+3+h)\ (h+1)}$		
$h>(6-w)$	$P_T^{\square\prime} = \frac{2W(1+I_T)^*}{(w+6+h)\ (h+1)}$	$P_T^{\square\prime} = \frac{4W(1+I_T)^*}{(w+15+h)\ (h+1)}$		
$h>2$ & $(h+1)<b$			$P_B^{\square\prime} = \frac{2W(1+I_B)}{(w+6+h)b}$	$P_B^{\square\prime} = \frac{4W(1+I_B)}{(w+15+h)b}$
$h>2$ & $(h+1)\geqq b$			$P_B^{\square\prime} \frac{2W(1+I_B)}{(w+6+h)\ (h+1)}$	$P_B^{\square\prime} = \frac{4W(1+I_B)}{(w+15+h)\ (h+1)}$

*Over portion of span covered.

against lateral deflection, and at times partly by passive resistance of earth back fill on the side opposite the one to which the excess lateral force is applied. Side sway problems will usually occur only in single barrels. In any double or triple barrel box large enough to consider from this standpoint, the top will be so stiff as a beam that the excess lateral force, even if applied on a small portion of the length, will be distributed almost uniformly over a considerable length of box. The maximum force to be taken by the sides for any one-foot section could then be determined directly.

It can be assumed that the bottom of the box does not move laterally. There will then be required an adjustment between the top, sides, and passive earth resistance so that the resisting load will bear the proper relation to the horizontal deflection of the top. The excess lateral pressure should be reduced to an equivalent force acting horizontally on the top slab of the box.

The box will usually have top and bottom slab of the same or nearly the same thickness. The two sides will usually have the same thickness but not necessarily the same thickness as the top and bottom. With these assumptions the sides will have the same moment (M) at the bottom and equal and opposite moments at the top, when a force is applied horizontally to the top slab. If it is assumed that forces to prevent rotation of the box are applied only at the corners so that no additional moments will be set up, the resulting deflection ($\triangle$) will be

Eq. 15—$\triangle = \dfrac{Mh}{6E}\left(\dfrac{b}{I_B}+\dfrac{h}{I_H}\right)$ or,

Eq. 16—$\triangle = \dfrac{Ph^2}{24E}\left(\dfrac{b}{I_B}+\dfrac{h}{I_H}\right)$ (for a single barrel box)

Where M = moments referred to above; P = force applied at top; E = modulus of elasticity; b and I_B, h and I_H, span length and moment of inertia of top and side respectively considering a one-foot section. These formulas give the force necessary to deflect the top with respect to the bottom, and considered in the opposite direction the force restraining the top against deflection as a horizontal beam.

There is a restraining force (F) at the top against lateral deflections due to the passive resistance of the earth, which if considered as a triangle would be

Eq. 17—$F = 1/3\ K_2 h \triangle$,

Where K_2 = pounds per square foot, per foot of deflection into earth; h = length of side span; $\triangle$ = deflection in feet of top laterally with respect to bottom. If the side is considered as having the shape of a guided cantilever the equation would read

Eq. 18—$F = 7/20\ K_2 h \triangle$, practically the same as Eq. 17.

The above assumptions are made for simplicity. There is, of course, an effect of moment due to the passive earth resistance of the side and bottom. It is suggested that these be taken into account only after an analysis shows that moments due to side sway are important from a design standpoint, for the reason that their consideration makes it difficult to compute a slope at the end of any member of the box as a starting point for deflection in the sides.

The resistance of the sides, and passive earth resistance where present, make up a combined force tending to restrain the top against horizontal deflection. These two forces can be combined to estimate the force causing side sway at some point along the box. The amount to be resisted by the sides can then be computed by the ratio of their resistance to that of the total for earth and sides, for the same deflection.

The adjustment of the top may produce a resisting force separate from the resisting forces of the sides and the passive earth resistance if it resists deflection through fixity imparted to it by cutoff walls or headwalls. The most general function of the top, however, in the side sway problem is to bend or deflect from its unloaded position in consequence of the excess lateral pressure and through this bending generate resisting forces in the sides from box stiffness and passive earth resistance as mentioned above. The top then is a beam loaded on one side by excess lateral pressure of a certain intensity, length, and location of load the same as for any other beam, and loaded on the other side by resisting forces which depend, for their magnitude at any point, upon the horizontal deflection of the beam at that point.

For the moment of inertia of the top against horizontal deflection the sides should be included with their full thickness at the top and tapering to zero at the bottom. Gross sections neglecting steel reinforcement can be used for computing the moments of inertia of members, using a modulus of elasticity of 720,000,000 lb. per sq. ft.

Several solutions for this problem have been offered, but as far as the writer knows, none for varying loads.

For a case where the lateral load is not uniform or is acting on only a portion of the length, the following equations will give an approximation of the distribution of the resisting load. Let

$\triangle_1$ = deflection of beam for known loads.

$\triangle_2$ = deflection of beam for loads resisting the deflection of the beam caused by known loads.

$\triangle = \triangle_1 + \triangle_2$

I_1 = moment of inertia of top slab against horizontal deflection in the case of side sway; or moment of inertia of the box against vertical deflection in the case of foundation reactions in relation to longitudinal moments.

R = load per lineal foot of box to cause a deflection on ($\triangle$) into the side earth combined with the resistance of the sides in bending; or into the foundation in connection with foundation reactions in relation to longitudinal moments.

K_R = coefficient such that $R = \triangle K_R$.

Resisting load $= -\triangle K_R$

$u = \dfrac{2\pi}{L}$ and $v = \dfrac{\pi}{L}$

w = total known load.

Then for resisting load only:

Eq. 19—Load $= a + \dfrac{bx}{L} + c \cos ux + d \sin vx$

Eq. 20—Shear $= ax + \dfrac{bx^2}{2L} + \dfrac{c}{u} \sin ux - \dfrac{d}{v} \cos vx + A$

Eq. 21—Moment $= \dfrac{ax^2}{2} + \dfrac{bx^3}{6L} - \dfrac{c}{u^2} \cos ux - \dfrac{d}{v^2} \sin vx + Ax + B$

Eq. 22—Slope $= \dfrac{1}{EI_1}\left[\dfrac{ax^3}{6} + \dfrac{bx^4}{24L} - \dfrac{c}{u^3} \sin ux + \dfrac{d}{v^3} \cos vx + \dfrac{Ax^2}{2} + Bx\right] + C_1$

Eq. 23—$\triangle_2 = \dfrac{1}{EI_1}\left[\dfrac{ax_4}{24} + \dfrac{bx^5}{120L} + \dfrac{c}{u^4} \cos ux + \dfrac{d}{v^4} \sin vx + \dfrac{Ax^3}{6} + \dfrac{Bx^2}{2}\right] + C_1x + D_1$

For the combined loads:

Eq. 24—$\left[\triangle_1 + \triangle_2\right] K_R + a + \dfrac{bx}{L} + c \cos ux + d \sin vx = 0$

For a fixed beam these equations are not suitable where the load is near one end. However, where the known load is near the center the maximum deflection will be practically at the center. For this case the deflections for known loads should be computed for a fixed beam with no deflection at the ends. The b term in the resisting load should then be omitted, and the constants computed for slope and deflection $= 0$ at $x = 0$ and $x = L$.

For a fixed beam, the resisting load for symmetrical loading will have constants in equation 23 as follows:

Eq. 25—$A = -\dfrac{aL}{2}$; $B = +\dfrac{aL^2}{12} + \dfrac{2d}{Lv^3}$; $C_1 = -\dfrac{d}{EI_1v^3}$; $D_1 = -\dfrac{c}{EI_1u^4}$

For a beam not fixed or supported at the ends the treatment would be as described later in connection with foundation reactions as effecting moments in a longitudinal direction.

For the usual case for fixed beam conditions where side sway would be considered, a portion of the span would be covered near the center by a load acting horizontally against the top slab. It will be necessary to determine the proportion of this load reststed by the sides against side sway.

Using Eq. 24 for $\triangle = 0$ at $x = 0$, $a = -c$.

Substituting in Eq. 24, this value for a, and the values of constants given in Eq. 25, we have two simple equations for two positions on the beam as follows, where deflections are in terms of $\frac{L_4}{EI_1}$

$$\text{Eq. 26—} x = \frac{L}{4}, \frac{K_R L^4}{EI_1}\left[-.002{,}106{,}46c + .001{,}212{,}01d + \triangle_{1\,x=\frac{L}{4}}\right] - c + .70711d = 0$$

$$\text{Eq. 27—} x = \frac{L}{2}, \frac{K_R L^4}{EI_1}\left[-.003{,}887{,}40c + .002{,}203{,}10d + \triangle_{1\,x=\frac{L}{2}}\right] - 2c + d = 0$$

The resisting load is determined by $\triangle_1$ and the value of $\frac{K_R L^4}{EI_1}$ as shown in Table 3.

TABLE 3—SIDE LOADS, FIXED BEAMS, SYMMETRICAL LOADING

	$\frac{K_R L^4}{EI_1}$	Length covered by w lbs. per lin. ft. at. top slab	Value of c	Value of d	Resistance load at $x=L/4$	Resistance load at $x=L/2$	Resistance load
1	1,000	.3 L	+ .33w	+ .24w	− .153w	− .42w	−.18wL
2	5,000	.3 L	+ .612w	+ .492w	− .264w	− .732w	−.314wL
3	12,500	.3 L	+ .6w	+ .446w	− .285w	− .754w	−.316wL
4	200,000	.3 L	+1.488w	+1.972w	− .094w	−1.005w	−.233wL
5	5,000	1.0 L	+.0077w	−1.069w	− .763w	−1.084w	−.674wL
6	50,000	1.0 L	− .716w	−2.436w	−1.007w	−1.004w	−.726wL

Items (3) and (4) represent a 6 x 6 ft. box with 9-in. top and 8-in. side. Passive earth resistance taken at 120,000 lbs. per square foot for 1 ft. deflection. The walls furnish 75 per cent of resistance load. Item (3) is 50 ft.; item (4), 100 ft. long. The items which show more total resistance load than the applied load indicate the imperfections of the formula. It is believed that the results are suitable for design purposes. The moments induced by the portion of resisting load taken by the sides can be obtained by placing moments on the sides as outlined by Cross

and Morgan[3]. The moments due to earth resistance would be included for total moments. The use of signs as proposed by Professor L. E. Grinter[4] will make it possible to add all plus and minus moments in the sides algebraically, in which case an excess of plus moments indicates a push on the top to the right.

LONGITUDINAL MOMENTS IN BOX

For the purpose of estimating the foundation reaction of a culvert or box, in order to estimate the longitudinal moments, equation 24 may be used in a manner similar to that used in determining side sway. In the case of foundation reactions a value of K_R is assumed as representing the resistance of the foundation against deflection. A value of K_R = 120,000 lb. per sq. ft. for a one-foot deflection represents a rather soft foundation. This value ranges up to high values for rock as given in some of the later text books. This value is sometimes called the "foundation modulus."

There has been considerable discussion as to whether a foundation would act in a stairght line manner as indicated above or whether it would vary with the applied load, and also with time. The important point is the relation of the foundation reaction along various parts of the box. Two probable values may be used for K_R and the design made or the worst condition shown by either. In case moments are indicated in excess of the moments which the box could take under working stresses, joints should be provided which would permit rotation and take shear.

An investigation of longitudinal moments in a culvert is necessary only where the foundation material is rather soft. A firm foundation would not permit the deflections which would necessarily be present in the case of important longitudinal moments. The question of longitudinal moments in a culvert or box structure will usually be covered by the case where the ends are not fixed or supported and where the load is not symmetrical.

For a beam not fixed and not supported at the ends, the resisting load should be considered as on a cantilever held at the right end with slope and deflection = 0 at left end ($x = 0$). The known loads, including any known moments, should be considered in the same manner. The deflections for both should be computed in terms of $\frac{L^4}{EI_1}$.

Then for a beam not fixed or supported at the ends equation 23 will have constants as follows:

Eq. 28—$A = +\frac{d}{v}$; $B = +\frac{c}{u^2}$

$$C_1 = -\frac{d}{v^3 EI_1}; \quad D_1 = -\frac{c}{u^4 EI_1}$$

Equation 24 is then written as:

Eq. 29 $$\frac{K_R L^4}{EI_1}\left[\triangle_1 + \triangle_2\right] + K_R C x + K_R D + a + \frac{bx}{L} + c\cos ux + d\sin vx = 0$$

Where C and D are the unknown slope and deflection at $x = 0$.

Values are required for a, b, c, d, $K_R C$, and $K_R D$.

Assume that the foundation reaction is equal and opposite to the weight of the structure and its top loads as shown in equation 30.

Eq. 30 $$-aL + \frac{bL}{2} + \frac{2d}{v} + W = 0$$

Assume that, taking moments about the right end, the moment of the foundation reaction is equal and opposite to the moment (Mw) caused by the structure and its top loads including any known moments, as shown in equation 31.

Eq. 31 $$\frac{aL^2}{2} + \frac{bL^2}{6} + \frac{dL}{v} + M_w = 0$$

From the above equations

Eq. 32 $$a = -\frac{6}{L^2}\left(M_w - \frac{WL}{2}\right) - \frac{W}{L} - \frac{2d}{vL} = Q - \frac{2d}{vL}, \text{ where}$$

$$Q = -\frac{6}{L^2}\left(M_w - \frac{WL}{2}\right) - \frac{W}{L}$$

Eq. 33 $$b = +\frac{12}{L^2}\left(M_w - \frac{WL}{2}\right)$$

From equation 29

Eq. 34 $$\text{at } x = 0;\ \frac{K_R L^4}{EI_1}\left[0\right] + K_R D + a + c = 0;\ K_R D = -a - c$$

$K_R C$ can be eliminated by subtracting from Eq. 29 for $x = L$, 4(Eq. 29) for $x = \frac{L}{4}$ and 2 (Eq. 29) for $x = \frac{L}{2}$.

Making substitutions for a and $K_R D$, (b being known), in Eq. 29, and eliminating $K_R C$ as indicated above, there remain two equations containing two unknowns c and d, where $\triangle_1$ is expressed in terms of $\frac{L^4}{EI_1}$

Eq. 35 $$\frac{K_R L^4}{EI_1}\Big[\underset{x=L}{\triangle_1} - 4\underset{x=L/4}{\triangle_1} + .041{,}015{,}62Q + .008{,}300{,}77b + .012{,}065{,}24c - .005{,}412{,}76d\Big] + 4c - 2.82844d = 0$$

$$\text{Eq. 36—}\frac{K_R L^4}{EI_1}\left[\underset{x=L}{\triangle_1} - \underset{x=L/2}{2\triangle_1} + .036{,}458{,}34Q + .007{,}812{,}49b + .008{,}898{,}98c - .003{,}954d\right] + 4c - 2d = 0.$$

In an actual problem the first four items in brackets in equations 35 and 36 will be combined into a single quantity.

For a structure similar to that shown in Figure I there will be a minimum load per lineal foot which can be deducted in order to reduce the size of quantities to be dealt with. This will usually be the weight of the box plus the minimum fill. The value of K_R will be multiplied by the width of the box in order to obtain the resistance per lineal foot of box.

Taking an actual case of culvert loads on a 3 x 3 ft. culvert 120 ft. long, equations 35 and 36 give results as shown in Table 4. A uniform load of 1,970 lb. which was in effect for the whole length was deducted.

TABLE 4—LONGITUDINAL MOMENTS FROM FOUNDATION REACTIONS (FEET AND POUNDS)

Item	$\frac{K_R L^4}{EI_1}$	Values					
			at $x=0$	at $x=L/4$	at $x=L/2$	at $x=3/4L$	at $x=L$
W		197,970					
L		120					
E		720,000,000					
K_R		480,000					
M_w			0	315,000	2,335,500		11,568,000
$\triangle_1$		$\left(\frac{L^4}{EI_1}\right)$	0	.085	2.40	15.	52.8
a		$\begin{cases} -1521 \\ -.63662d \end{cases}$					
b		-258.5					
c	7960	$+2815$					
d	7960	$+4040$					
Resisting load	7960		-1280	$-1,300$	$-3,000$	-1430	$-1,540$
Resisting moment	7960		0	$-363,000$	$-2,026,500$		
Net moment	7960		0	$-48,000$	$+309,000$		

Table 4 indicates a moment at the center of +309,000 foot pounds, indicating tension in the top slab as W was considered positive. The maximum indicated foundation load is 3,000 lb. per lin. ft. at the center.

A uniform load of 1,970 lb. had been deducted before Table 4 was computed, so the maximum foundation reaction would be (3000 + 1970) or about 5,000 lb. per lin. ft. of box. If the foundation material will support this load with practically no settlement, there would be no necessity for extra reinforcing steel or joints.

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The examples cited herein have come up in connection with work done by the Bureau of Reclamation. However, no statement or opinion given here should be taken as expressing the views or policy of the Bureau.

Plans and designs of canal structures are under the supervision of H. R. McBirney, Principal Senior Engineer in charge of canals, and J. L. Savage, Chief Designing Engineer. All activities of the Bureau are under the direction of S. O. Harper, Chief Engineer, and John C. Page, Commissioner.

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Discussion of a paper by C. R. Burky:

The Design of Box Culverts*

By DALE H. REA and AUTHOR

By DALE H. REA†

Mr. Burky has presented a unique graphical method for finding the transverse moments in one and two cell box culverts, but states that moment distribution should be used for a solution of box culverts of more than two cells. The writer discovered a graphical method some years ago that precludes the tediousness of indeterminate analyses of multiple-cell box culverts. The method consists of formulas and a graph for evaluating the unknowns in the formulas.

ASSUMPTIONS

In a multiple-cell culvert the clear span, clear height, thickness of the top slab, and the thickness of the bottom slab of one cell are equal to those of the other cells. This allows the thickness of the piers and ends, and the thickness of the top and bottom slabs, to be different. These assumptions follow the usual practice and give the most economical culvert.

The intermediate piers are assumed to take no bending stress. This assumption simplifies the solution greatly and introduces only a small error. Under average conditions the pier moment seldom exceeds four per cent of the adjacent floor slab. The thickness of piers is usually determined by construction requirements.

A section of the culvert one foot long is chosen for convenience of analysis. Since only relative values of moments of inertia are needed, they may be expressed as the cube of the depth of the beam.

The system of signs is based on the assumption that a horizontal beam with loads acting downward give a positive fixed end moment at the left and a negative fixed end moment at the right of the beam. The signs of the formulas shown in Table A conform to this assumption and refer to

*ACI JOURNAL, Sept. 1942; *Proceedings* V. 39, p. 33.
†Assistant Engineer, Bureau of Reclamation, Denver, Colo.

TABLE A—FINAL END MOMENT FORMULAS

ONE-CELL BOX

$$M_A = +h^2 (C_1W_1 + C_2W_2) + L^2(C_4W_3 - C_5W_4)$$
$$M_B = -h^2 (C_1W_1 + C_3W_2) + L^2(C_5W_3 - C_4W_4)$$

TWO-CELL BOX

$$M_A = +h^2 (C_1W_1 + C_2W_2) + L^2(C_4W_3 - C_5W_4)$$
$$M_B = -h^2 (C_1W_1 + C_3W_2) + L^2(C_5W_3 - C_4W_4)$$
$$M_C = -\frac{h^2}{2}\left(C_1W_1 + C_2W_2\right) + L^2\left(C_6W_3 + \frac{1}{2}C_5W_4\right)$$
$$M_D = +\frac{h^2}{2}\left(C_1W_1 + C_3W_2\right) - L^2\left(\frac{1}{2}C_5W_3 + C_6W_4\right)$$

THREE-CELL BOX

$$M_A = +h^2 (C_1W_1 + C_2W_2) + L^2(C_4W_3 - C_5W_4)$$
$$M_B = -h^2 (C_1W_1 + C_3W_2) + L^2(C_5W_3 - C_4W_4)$$
$$M_C = -\frac{h^2}{5}\left(C_1W_1 + C_2W_2\right) + L^2\left(C_6W_3 + \frac{1}{5}C_5W_4\right)$$
$$M_D = +\frac{h^2}{5}\left(C_1W_1 + C_3W_2\right) - L^2\left(\frac{1}{5}C_5W_3 + C_6W_4\right)$$

FOUR-CELL BOX

$$M_A = +h^2 (C_1W_1 + C_2W_2) + L^2(C_4W_3 - C_5W_4)$$
$$M_B = -h^2 (C_1W_1 + C_3W_2) + L^2(C_5W_3 - C_4W_4)$$
$$M_C = -\frac{2}{7}h^2\left(C_1W_1 + C_2W_2\right) + L^2\left(C_6W_3 + \frac{2}{7}C_5W_4\right)$$
$$M_D = +\frac{2}{7}h^2\left(C_1W_1 + C_3W_2\right) - L^2\left(\frac{2}{7}C_5W_3 + C_6W_4\right)$$
$$M_E = +\frac{1}{7}h^2\left(C_1W_1 + C_2W_2\right) + L^2\left(C_7W_3 - \frac{1}{7}C_5W_4\right)$$
$$M_F = -\frac{1}{7}h^2\left(C_1W_1 + C_3W_2\right) + L^2\left(\frac{1}{7}C_5W_3 - C_7W_4\right)$$

FIVE-CELL BOX

$$M_A = +h^2(C_1W_1 + C_2W_2) + L^2(C_4W_3 - C_5W_4)$$
$$M_B = -h^2 (C_1W_1 + C_3W_2) + L^2(C_5W_3 - C_4W_4)$$
$$M_C = -\frac{5}{19}h^2\left(C_1W_1 + C_2W_2\right) + L^2\left(C_6W_3 + \frac{5}{9}C_5W_4\right)$$
$$M_D = +\frac{5}{19}h^2\left(C_1W_1 + C_3W_2\right) - L^2\left(\frac{5}{19}C_5W_3 + C_6W_4\right)$$
$$M_E = +\frac{1}{19}h^2\left(C_1W_1 + C_2W_2\right) + L^2\left(C_7W_3 - \frac{1}{19}C_5W_4\right)$$
$$M_F = -\frac{1}{19}h^2\left(C_1W_1 + C_3W_2\right) + L^2\left(\frac{1}{19}C_5W_3 - C_7W_4\right)$$

SIX-CELL BOX

$$M_A = +h^2 (C_1W_1 + C_2W_2) + L^2 (C_4W_3 - C_5W_4)$$
$$M_B = -h^2 (C_1W_1 + C_3W_2) + L^2 (C_5W_3 - C_4W_4)$$
$$M_C = -\frac{7}{26}h^2\left(C_1W_1 + C_2W_2\right) + L^2\left(C_6W_3 + \frac{7}{26}C_5W_4\right)$$
$$M_D = +\frac{7}{26}h^2\left(C_1W_1 + C_3W_2\right) - L^2\left(\frac{7}{26}C_5W_3 + C_6W_4\right)$$
$$M_E = +\frac{2}{26}h^2\left(C_1W_1 + C_2W_2\right) + L^2\left(C_7W_3 - \frac{2}{26}C_5W_4\right)$$
$$M_F = -\frac{2}{26}h^2\left(C_1W_1 + C_3W_2\right) + L^2\left(\frac{2}{26}C_5W_3 - C_7W_4\right)$$
$$M_G = +\frac{1}{26}h^2\left(C_1W_1 + C_2W_2\right) + L^2\left(C_8W_3 + \frac{1}{26}C_5W_4\right)$$
$$M_H = -\frac{1}{26}h^2\left(C_1W_1 + C_3W_2\right) - L^2\left(\frac{1}{26}C_5W_3 + C_8W_4\right)$$

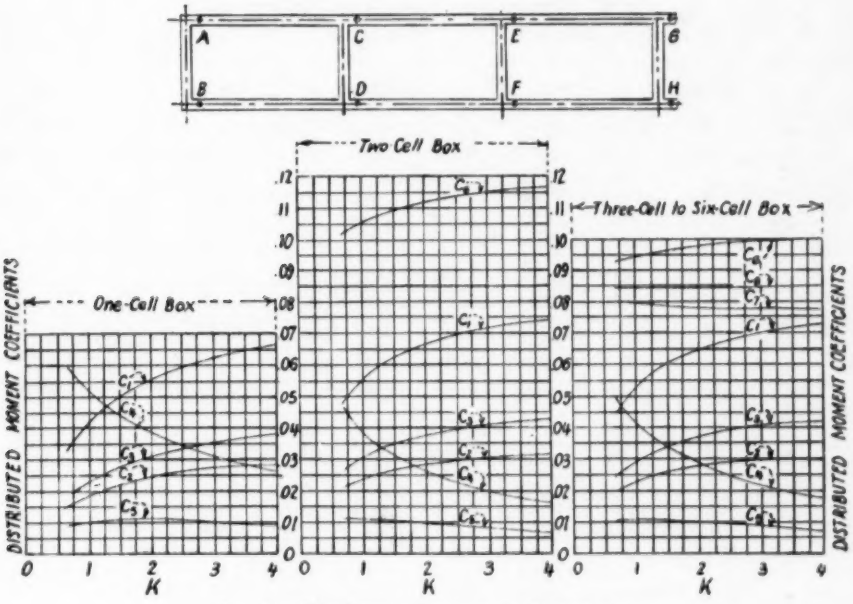

Fig. A—Distribution moment coefficient curves

the end of the member indicated on Fig. A. The moments of the connecting member will be of opposite sign but of equal numerical value.

The loading on the box must be symmetrical about the vertical centerline to preclude sidesway. With a symmetrical structure and loading, all moments will be symmetrical about the vertical centerline. Therefore, it is only necessary to consider moments of half of the box culvert.

A triangular, uniform, or trapezoidal loading may be used at the ends of the box. They may be used as water loads acting in the opposite direction if the signs of the affected coefficients are changed in the formulas. Loads on the top and bottom slabs are uniform or may be replaced by an equivalent uniform load. An equivalent uniform load is one that will produce the same fixed end moments in the member as that produced by the assumed loading.

Experience has shown that some of the above assumptions may be modified without materially affecting the results. However, the vast majority of concrete culverts can be designed by this method without revision.

NOTATION

C_1, C_2 .... C_8 distributed moment coefficients.
t, thickness of beam.
h, height, center to center, between horizontal members.
I, relative moment of inertia of any member = $(t)^3$.

TABLE B—DISTRIBUTED MOMENT COEFFICIENTS

K	No. of Cells	C_1	C_2	C_3	C_4	C_5	C_6	C_7	C_8
	1	.0333	.0151	.0182	.0591	.0091			
	2	.0476	.0212	.0264	.0467	.0110	.1016		
0.667	3	.0438	.0197	.0242	.0501	.0107	.0900		
	4	.0448	.0200	.0246	.0492	.0108	.0930	.0785	
	5	.0446	.0199	.0246	.0495	.0107	.0922	.0816	
	6	.0444	.0199	.0245	.0496	.0107	.0926	.0800	.0842
	1	.0417	.0188	.0229	.0521	.0104			
	2	.0555	.0244	.0311	.0389	.0111	.1056		
1.000	3	.0521	.0231	.0290	.0424	.0112	.0915		
	4	.0531	.0235	.0296	.0415	.0112	.0953	.0773	
	5	.0528	.0233	.0294	.0417	.0112	.0943	.0811	
	6	.0527	.0233	.0293	.0419	.0112	.0948	.0792	.0844
	1	.0500	.0222	.0278	.0444	.0111			
	2	.0625	.0271	.0354	.0313	.0104	.1094		
1.500	3	.0595	.0260	.0336	.0347	.0108	.0931		
	4	.0603	.0263	.0341	.0337	.0107	.0975	.0762	
	5	.0601	.0262	.0339	.0339	.0105	.0963	.0807	
	6	.0600	.0262	.0339	.0341	.0107	.0969	.0785	.0846
	1	.0583	.0255	.0328	.0359	.0109			
	2	.0687	.0292	.0394	.0236	.0090	.1132		
2.333	3	.0663	.0284	.0379	.0266	.0096	.0947		
	4	.0672	.0288	.0384	.0258	.0097	.0996	.0751	
	5	.0669	.0286	.0382	.0260	.0095	.0984	.0803	
	6	.0668	.0286	.0382	.0261	.0096	.0990	.0777	.0848
	1	.0667	.0286	.0381	.0261	.0095			
	2	.0741	.0310	.0431	.0160	.0067	.1170		
4.000	3	.0725	.0305	.0420	.0183	.0075	.0963		
	4	.0728	.0305	.0423	.0177	.0073	.1021	.0740	
	5	.0728	.0306	.0422	.0179	.0073	.1004	.0799	
	6	.0727	.0305	.0422	.0180	.0074	.1012	.0770	.0851

K, ratio of stiffness of horizontal member to vertical member $= \frac{I}{L} \div \frac{I}{h}$

L, span, center to center, between vertical members.

M_A, M_B .... M_H, final end moments at A, B, ... H.

W_1, uniform load on end of box in lb. per lin. ft.

W_2, triangular load on end of box in lb. per lin. ft.

W_3, uniform load on top of box in lb. per lin. ft.

W_4, uniform load on bottom of box in lb. per lin. ft.

FORMULAS

By applying influence loads to multiple-cell culverts of various stiffness ratios, and distributing moments seven times, the coefficients shown in Table B were derived. They were used to plot the curves in Fig. A and are listed so that curves of larger scale may be plotted by anyone.

The final end moment formulas shown in Table A are for boxes of one to six cells. They are composed of a maximum of fourteen functions.

There are four load functions, two span functions, and five to eight coefficient functions. The load and span functions are ascertained from the conditions of a given problem. The coefficients are taken from Fig. A once the culvert shell has been chosen. An example will best illustrate the use of the formulas. Given: 12 ft.-0 in. x 13 ft-6 in. three-cell box culvert

Uniform end load, $W_1 = 338$ lb.
Triangular end load, $W_2 = 62.5 \times 14.54 = 909$ lb.
Uniform top load, $W_3 = 771$ lb.
Uniform bottom load, $W_4 = 921$ lb.

Assume: Top slab $t = 12$ in. End slab, $t = 13$ in.
Bottom slab, $t = 13$ in. Piers, $t = 10$ in.

For the top slab: $K = \dfrac{(12)^3}{12.96} \div \dfrac{(13)^3}{14.54} = .88$

For the bottom slab: $K = \dfrac{(13)^3}{12.96} \div \dfrac{(13)^3}{14.54} = 1.12$

Entering Fig. A with the known stiffness ratios, the coefficients are obtained and substituted in the formulas of Table A for a three-cell box thus:

$$M_A = +(14.54)^2(.0496 \times 338 + .0218 \times 909) + (12.96)^2(.0444 \times 771 - .0110 \times 921) = +11{,}770 \text{ ft. lb.}$$

$$M_B = -(14.54)^2(.0540 \times 338 + .0305 \times 909) + (12.96)^2(.0110 \times 771 - .0400 \times 921) = -14{,}480 \text{ ft. lb.}$$

$$M_c = -\frac{(14.54)^2}{5}(.0496 \times 338 + .0218 \times 909) + (12.96)^2\left(.0910 \times 771 + \frac{.0110 \times 921}{5}\right) = +10{,}600 \text{ ft. lb.}$$

$$M_D = +\frac{(14.54)^2}{5}(.0540 \times 338 + .0305 \times 909) - (12.96)^2\left(\frac{.0110 \times 771}{5} + .0910 \times 921\right) = -12{,}410 \text{ ft. lb.}$$

AUTHOR'S CLOSURE

Mr. Rea, in his discussion, has given a set of coefficients for end moments for boxes of from one to six cells, arrived at by moment distribution. The moments in the intermediate piers were neglected and the load was assumed to be uniform on the top and bottom.

It would be well to repeat the statement in the original paper referred to by Mr. Rea. "For more than 2 barrels, or where loading or box is not

symmetrical about the vertical centerline, it is better to use the Hardy Cross method of moment distribution than to attempt to set up formulas." That is, for two barrels, simple formulas may be derived, regardless of whether the load is uniform over the top and bottom. Any loading can be put on the top and bottom as long as the loading is symmetrical about the vertical centerline of the structure. Several assumptions as to foundation reaction may be made.

For ordinary one-barrel or two-barrel boxes, say up to five or six feet in height, it is usually sufficient to assume a uniform foundation reaction. This will give large moments in the bottom slab. If the foundation reaction is concentrated near the ends of the cells, the resulting final end moments in the side slab will be smaller at the bottom and larger at the top, than for uniform foundation reaction. However, this larger moment at the top of the side slab is usually not greater than the moment at the bottom for uniform foundation reaction. The same bar is usually taken up the side to furnish the required steel for negative moment at both the top and bottom of the side slab. For larger boxes of several cells, one might wish to investigate different distributions of the foundation reaction.

With more than two cells, a greater number of arbitrary assumptions are necessary in order to derive simple formulas for the final end moments. One might get the impression from Mr. Rea's comments that the coefficients given by him can be used if the loading is symmetrical about the vertical centerline. To use these coefficients, the loading on top or bottom must be the same on each cell and symmetrical about the vertical centerline of each cell; also, the intermediate vertical walls must be considered as having no stiffness.

A complete check of the problem given by Mr. Rea, on the assumptions used by him, was made by moment distribution in 25 minutes. This included computation of fixed-end moments, distribution of moments in accordance with relative stiffness, and algebraic addition at the joints to get final end moments. The moments given by Mr. Rea were checked within 1½ per cent. The method given by Mr. Rea would be useful in the investigation of a change of one condition, the other remaining the same.

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Welding of 2-Inch Square Reinforcing Bars for Piers of Pit River Bridge*

By ROBERT SAILER†

SYNOPSIS

Two-inch square reinforcing bars, 60 feet long, were used for the main reinforcement in the tall concrete piers of the Pit River Bridge. Since more than 8,000 splices, laps or welds, were required laboratory tests were made to determine whether adjoining bars could be satisfactorily welded.

Results of these tests, prior to construction, showed that satisfactory welds could be obtained if low-carbon steel was used and if certain precautions were taken in the welding operations. The specifications for the steel and the necessary precautions are discussed. Tests of welds during construction confirmed the results of the laboratory tests.

The Pit River Bridge provides for four lanes of highway traffic and two main-line railroad tracks across one of the arms of the reservoir created by Shasta Dam. The superstructure of this bridge is supported by unusually tall concrete piers, each of the two central piers being 360 ft. high. As far as known, they are the highest piers in the world. When the reservoir is filled, however, the piers will be almost entirely submerged, and the pier tops will be a few feet only above the water level of the reservoir.

Since the piers were designed to resist earthquakes as well as wind and braking and sway forces from railroad loading, an unusually large amount of reinforcement was required. Calculations showed that for some sections the vertical reinforcement at each face of the pier shaft would have to consist of as many as eight rows of 1¼-in. square bars spaced 4½-in. on centers. As eight rows of bars spaced so closely would make proper placing of concrete extremely difficult, it was decided to use 2-in. square bars spaced not closer than 6-in. centers. Thus it was possible to reduce the number of rows from eight to four.

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†Bureau of Reclamation, Denver.

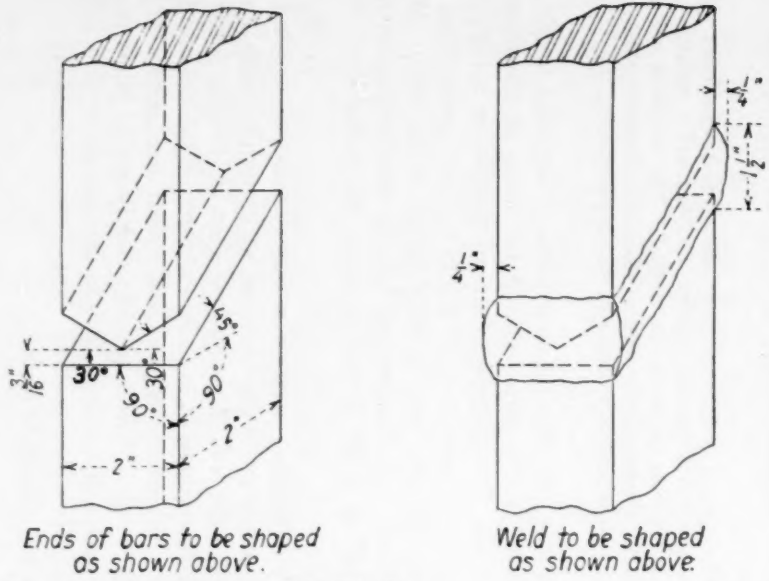

Fig. 1 (above)—Detail of weld for 2-in. bars

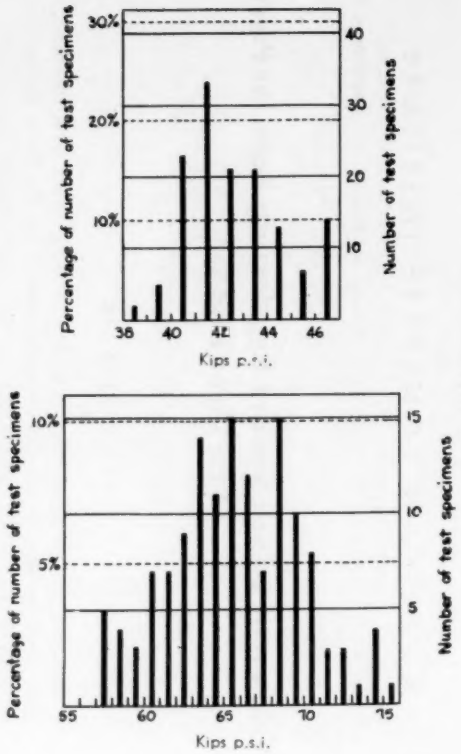

Fig. 2 (at right)—Variations in strengths and number of test specimens reaching such strengths: at top, yield point in kips per square inch; below, strength in kips per square inch

Even though the 2-in. square bars were placed at 6-in. centers (4 in. in the clear), it would still be difficult to place concrete at the splices if the bars were lapped. Moreover, a total of about 8,200 splices were required. Investigations were therefore made to determine the practicability of welding bars at the splices so as to eliminate lapping.

Only limited data were found on welding of reinforcing bars, and it was decided to have tests made in the laboratory of the Denver Office of the Bureau of Reclamation. Results of these tests, together with experiences on other welded construction works, showed that certain precautions in welding the bars should be taken to insure suitable welds with adequate strengths.

SPECIAL PROVISIONS

The special provisions embodied in the specifications are summarized and briefly discussed as follows:

1. The reinforcement bars should be of steel having as low a carbon content as compatible with strength requirements. The minimum yield point strength was specified as 40,000 psi, the minimum ultimate strength as 70,000 psi; and the maximum carbon content was specified as 0.35 per cent.

2. The ends of the bars should be shaped as shown in Fig. 1 by either sawing, machining, or torch cutting, provided that in case the latter method is used the effects of oxidation should be removed by grinding. Various types of end cuts were investigated, but the limited number of tests did not reveal differences in strengths. It was observed, however, that a flat surface for the lower bar and "V" cut on the upper bar as shown in Fig. 1 seemed to facilitate the placing of the weld material.

3. The adjoining ends of the bars should be preheated to a temperature of 500 F. for a distance of 9 in. and this temperature should be maintained during the welding.

4. Welding rods should be of the heavily coated, all-position type.

5. Before being given permission to weld, operators should be required to make two test welds, which when tested should exhibit yield points and breaking points not lower than 80 per cent of those specified for the material. The qualifying tests showed that only a small number of welders could meet the requirements of the specifications. It is believed that the demand for welders in nearby shipyards was greatly responsible for the difficulty in securing a larger number of expert welders.

6. To make sure that proper welding technique is maintained during the progress of the work, specimens for testing should be obtained by cutting welded splices out of the bars placed in the pier. One test should be made for each 100 welds completed on the job by each operator.

MATERIAL

In regard to the first requirement, the Columbia Steel Co., which furnished the material, cooperated with the Bureau in keeping the carbon content as low as compatible with strength requirements, and tests showed that the steel generally had a carbon content of 0.32 per cent and never more than 0.35 per cent.

It has been observed that standard commercial steel bars of intermediate grade steel have carbon contents from 0.42 per cent to 0.45 per cent and exhibit strengths about 20 per cent above the minimum requirements of standard specifications. Tests showed that when bars of such standard material were welded the strength of the weld was consistently below that of the material. The tests also showed that while the strength at the yield point was uniform for the various specimens tested, the ultimate strength varied a great deal and was often only a few thousand pounds above that of the yield point.

In contrast to standard commercial bars, the specially rolled bars for the Pit River bridge, which had carbon contents not exceeding 0.35 per cent, showed no such irregular strengths. The welds equaled the minimum strength at the yield point of the material, ran short a few per cent of the minimum ultimate strength, but showed a high degree of uniformity for all tests, though made by different welders and at different times during the construction. Therefore, it would seem advisable that if electric welding of bars becomes more general, the specifications should limit the carbon content to not more than 0.35 per cent, particularly if the higher strength of material now commonly furnished is not utilized in the design because of adherence to design unit stresses recommended by various codes.

It was anticipated that the special requirements combined with the special rolling of 2-in. square bars would increase the price somewhat. However, the unit price as contracted was found to be the same as for standard reinforcement. It is believed that the large quantity, amounting to 4,350 tons, largely contributed to lowering the price. If small quantities are ordered, some increase in price must be expected.

WELDING OPERATIONS

To keep adjoining bars, which were 60 feet long, in proper alignment during the welding operations the contractor used a steel clamp. This clamp, provided with a copper backing strip to prevent fusion, was fastened to the lower bar with two tapered pins and to the upper bar with one tapered pin passing through U-shaped yokes. (Fig. 3). In placing the bars, a key piece was inserted between the ends of the bars so as to insure the maintenance of $\frac{3}{16}$-in root for the weld as required by the specifications.

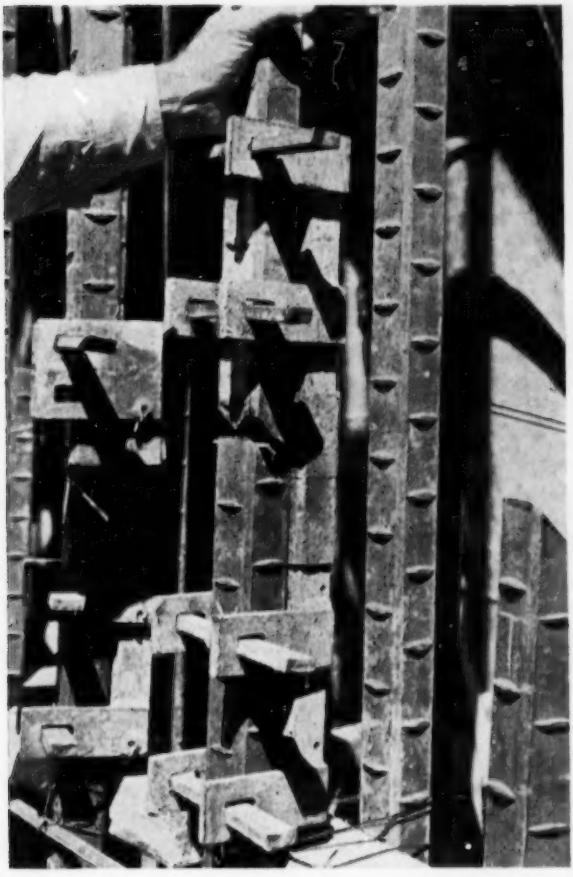

Fig. 3—Detail of clamp holding 2-in. square reinforcing bars in position for welding

Butane gas applied through a double jet was used to preheat the ends of the bars to a 500 F., which could be readily tested with 50-50 lead solder. See Fig. 4. The welding was then started. The first two passes joined the apex of the "V" of the upper bars with the flat surface of the lower bar. Further depositing proceeded evenly on both sides, peening following every pass. Una No. 2500, Fleetweld No. 5 and No. 7 welding rods were used, all with equally satisfactory results. As an average, 50 minutes were required to complete one entire welded splice. The contractor, who furnished the welding rods, was paid $2.20 for each weld.

CONCLUSION

In conclusion, it may be stated that the welding of bars at splices as compared with lapping is entirely practicable, and reliable results can be obtained provided precautions are taken as outlined. For the Pit River

Fig. 4—Butane heating apparatus used to preheat steel bars for 9 in. on either side of weld.

Bridge, the welding of the bars proved to be more economical than lapping. On the basis of the contract prices $28,000 was actually saved by welding.

ORGANIZATION

The designs and specifications for the welding of bars were prepared in the Denver Office of the Bureau of Reclamation, under the direction of the writer and under the supervision of H. R. McBirney, Principal Senior Engineer of Canals, and J. L. Savage, Chief Designing Engineer. Construction of the Pit River Bridge was supervised by Ralph Lowry, Construction Engineer of the Kennett Division, Central Valley Project. All activities of the Bureau are under the direction of S. O. Harper, Chief Engineer, and John C. Page, Commissioner. The Bureau of Reclamation is in the Department of the Interior, Harold L. Ickes, Secretary. The Union Paving Co. of San Francisco, Calif., was the contractor for the substructure of the bridge.

Discussion of this paper should reach the ACI Secretary in triplicate by March 1, 1943 for publication in the JOURNAL for June, 1943

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Job Problems and Practice

WHAT JPP MEANS

—an informal clearing house for mutual help on concrete problems; an opportunity for the ACI member who *knows* to tell the ACI member who *needs* the information. If the subject matter holds probabilities of general interest it will be briefed in these pages—with authorship credit unless the contributor prefers not.

If you know of an interesting problem whose solution has smoothed the way of someone in the field, record it briefly. If you have a problem, present it; a question, ask it. If you are moved to helpful comment or constructive criticism write the Secretary.

The "answers" are those of individuals to whom the questions are referred and not of the ACI.

Why Grinding Aids (39-106)*

Q—What is the best explanation of the effects of grinding aids on portland cement concrete?

BY C. H. SCHOLER†

A—It has been amply demonstrated that the use of additions interground with the cement which cause the inclusion of small quantities of air in the cement and water paste, gives a marked increase in the resistance to freezing and thawing of concrete made with such paste.

Most but not all of the common grinding aids tend to cause the inclusion of air in the paste. In so far as the author has any information, grinding aids which do not cause air inclusions do not promote increased durability in concrete made with such cement.

That the air inclusions are the primary cause of such increased durability has been shown by various tests.‡

Hand mixed concrete using such cements mixed with the least possible amount of energy and containing but little entrained air in the paste, do

*From the 1942 ACI Convention "Quiz Session."
†Kansas State College, Manhattan, Kan.
‡See A. A. Anderson, "Treated Cement Concrete Resists Scaling; 17th Annual Convention, Association of Highway Officials of the North Atlantic States, Boston, February, 1941.

not have the increased resistance to frost action found in identical mixes vigorously mixed mechanically and containing considerably more entrained air.

Air inclusions caused by other means such as the use of hydrogen peroxide caused a similar increase in resistance to freezing and thawing action.

Mr. Anderson indicates that the best results were secured with air contents between 3 and 6 per cent by volume.

It is probable that the increased durability is due to the following reasons:

1. A reduction in the saturation coefficient. That is, the relation between the volume of water absorbed and the total pore space in the concrete.

2. A reduction in the volume of water absorbed by the concrete.

3. The cushioning effect of the air bubbles adjacent to the layers of ice formed as the free water in the concrete freezes.

4. The freedom from segregation which should result from the cohesive highly plastic concrete produced by most such treated cements.

Concrete Tanks for Boiler Feed Water (39-107)

Q—Is it desirable to use concrete tanks for steam condensate and would a lining of wood pulp for such tanks be desirable?

A—Concrete has been used successfully for the construction of many tanks for steam condensate. Such tanks are subjected to two unusual conditions which must be carefully considered in the design and construction. The great range in temperature to which the tank will be subjected calls for an especially careful structural design which will consider the stresses due to temperature variation. Unquestionably, the very pure water is much harder on the concrete than water which contains some dissolved salts.

This pure water has a tendency to remove soluble salts from the concrete. The chief constituent which will be removed is calcium hydroxide. However, if the concrete is well made and properly cured so as to produce an impermeable structure, the solvent action will be confined to the exposed surface and there should be no concern regarding disintegration through removal of soluble constituents. After the concrete has been thoroughly cured it should be allowed to dry for a week or more for the occurrance of carbonation of any free lime on the surface.

It would be expected that very little calcium hydroxide and practically no silicates would go into solution. The successful use of concrete tanks for this purpose would seem to substantiate this.

I have not heard of coating concrete tanks with wood pulp and wonder how this could be applied to make it stick where continuously immersed in water.

If the problem is with an existing tank in which the quality of the concrete is not the best and some surface treatment is necessary, the use of a high temperature bituminous enamel might be considered. A linseed oil treatment should also prove satisfactory if the temperatures are not too high—say not more than 150° F.

How High a Lift of Concrete at One Placing (39-108)

Q—Is there any limit to the height which can be placed as one lift, one day's work, of a thin reinforced concrete wall? For example, would 8 ft. be too high for a wall 8 in. thick?

BY J. W. KELLY*

A—The maximum permissible height of lift is logically governed by the effects of water gain, and cannot be stated arbitrarily. In a thin reinforced concrete wall, which requires a fairly wet mix for placement, water gain is inevitable regardless of excellence of proportioning. The ill effects of water gain are dilution of the upper portions of the concrete and laitance at the surface of the lift. Dilution would cause increased shrinkage and decreased weathering resistance. Within reasonable limits, however, dilution does not seem to cause additional cracking or deterioration, as evidenced by the condition of many walls which have been placed for a full story height. The only specific recommendation of which the writer is aware is for architectural concrete walls, as follows: "A reasonable placing rate should be considered, generally not to exceed 2 feet an hour, and a lift of about 6 feet between joints should be a maximum."†

Lessening the water content of the concrete near the top of the lift is sound in principle, but it is difficult to apply in practice.

BY R. B. YOUNG‡

A—There is no limit to the height to which concrete can be placed in one continuous operation except the limits imposed by forms and the capacity of mixing or placing equipment available. Concrete has been placed to heights of 80 ft. without horizontal joints by the Hydro Electric Power Commission of Ontario** without detriment to the concrete. Some of the concrete placed in this way has been in service for more than ten years and its quality proven by time.

For a thin reinforced wall, the height to which concrete could be placed in one operation would probably be determined by the forms, particu-

*University of California, Berkeley.
†"Architectural Concrete for Small Buildings," Portland Cement Association, Chicago, 1937, 36 pp.
‡Hydro-Electric Power Com. of Ontario, Toronto.
**ACI JOURNAL, Feb. 1933; *Proceedings* Vo. 29, p. 249, "Concreting Problems—Chats Falls Power Development," by H. L. Trotter and W. Schnarr.

larly the cost of building a form strong enough to contain concrete built up at the rates common to this type of construction. Then, in walls as thin as 8 in. there is a limit to the depth of forms in which it is possible to get at the concrete to consolidate it properly. It should be possible to place an 8-in. wall in lifts of 8 to 10 ft. and 12 to 15 in. walls in lifts up to 15 ft.

BY LEWIS H. TUTHILL*

There are two factors which may affect the quality of concrete placed in a high wall in one lift, and the prominence of these factors in turn controls the height of lift which may be placed without obtaining inferior concrete in the upper portion. These factors are the water content of the concrete and the tendency toward bleeding which increases water content toward the top of a lift. Usually both may be controlled to some extent on the job. Finer cement, more cement, more fines in sand, and certain admixtures will all contribute to a reduction in bleeding. The water content usually can be reduced as the more accessible upper portions of the wall are reached. Usually, however, this reduction in water content (and slump) is not carried as far as would be practicable if more serious attention were directed toward properly timing the arrival of such concrete at the forms, and to the provision for transporting and placing facilities capable of handling the lowest slump concrete which could be properly worked in the upper forms by vibrators.

It is of course assumed that suitable facilities are at hand for placing and working the concrete at the bottom of a deep narrow wall. To keep overall settlement at a minimum, the concrete should be placed in layers not exceeding 12 in. deep and with as much time between them as is possible without introducing danger of cold joints. Frequently placed flexible dropchutes are required or a sufficient number of outside drops and ports arranged to prevent separation as the concrete enters—either or both supplemented by effective vibration. With such methods and facilities for placing, and a concrete of minimum bleeding tendency and water content, the maximum lift of 8 ft. suggested for an 8 in. wall would not be too high. In fact the concrete in many transition walls, flood channel walls, and in walls of about this thickness for other purposes has been placed in one lift to depths several times eight feet. If the rate of placing of suitable concrete is properly timed, there is no limit, from the standpoint of the concrete, to the height of a wall that can be placed in one lift. At such a placing rate, form pressures do not increase beyond the maximum reached with 4 to 6 foot depth.

It might be helpful to review Part IV of the report of ACI Committee 614 Recommended Practice of Measuring, Mixing and Placing of Con-

*Bureau of Reclamation, Denver, Colo.

crete* and pertinent sections of the Bureau of Reclamation Concrete Manual, third edition. On page 11 of the ACI Manual of Concrete Inspection, there is an excellent discussion of settlement which is also pertinent.

What Contraction Joint in a Reinforced Concrete Wall (39-109)

Q—What would be the maximum advisable length between contraction joints in a reinforced concrete wall 8 in. thick, if cracking is to be avoided?†

BY J. W. KELLY‡

A—The answer depends on many factors. Is it a retaining wall, which will be continuously damp on one side, or a building wall? If it is a building wall, will the building be heated? Will the wall be exposed or will it be veneered with brick or stucco? When will it be built, in hot or cold weather as compared to the average annual temperature? What is the likely temperature range in service, and the likely duration of the annual drying season? Is it exposed to the midday sun? Is the concrete of such a mix, and does it contain such materials, that the drying shrinkage will be relatively high? What is the percentage and distribution of reinforcement? Even if definite information were furnished on all of these items, prediction of the interval between cracks (as an indication of proper joint spacing) would be only a rough guess.

On the few occasions where authorities have ventured into print on this subject, they have relied principally on experience. The 1940 Joint Committee Report recommends for reinforced concrete retaining walls that vertical construction joints with V-notches at the face be provided at sections not over 30 ft. apart, with reinforcement carried through the joint. The committee recommends (with reservations) expansion joints in walls of long buildings at intervals of 200 to 300 ft., but does not prescribe any spacing for contraction joints.

The Bureau of Reclamation has observed, on experimental sections of 6-in. canal lining in arid climate, that the more or less random transverse cracking associated with continuous longitudinal reinforcement may be largely eliminated by concentrating the shrinkage at transverse joints spaced about 20 ft. apart.**

For the exposed walls of architectural concrete buildings, vertical weakened plane contraction joints have been successfully used for several years. William T. Wright recommends that these be placed at openings on either side of wide sections of wall, in all panels between

*ACI JOURNAL, Nov. 1941; *Proceedings* Vo. 38, p. 93—since adopted as an ACI Standard.
†From an ACI member in Australia.
‡University of California, Berkeley, Calif.
**"Concrete Manual," U. S. Bureau of Reclamation, Denver, 454 pp.

pilasters thicker than the intervening panel walls, and in large wall areas at intervals of 15 to 25 ft.*

BY LEWIS H. TUTHILL†

A 20 ft. maximum length of wall between joints, if cracking is to be avoided, is generally good. However, if a wall is less than 10 ft. high, and restrained at the foundation by anchorage to a more massive structure or a rock foundation, it would probably crack sooner or later near the center of a 20-ft. section. Therefore, joints at 10 ft. intervals would be advisable in walls of this or lesser height. For wall heights between 10 and 20 feet, the spacing of joints should roughly equal the height. No wall should be divided in lengths greater than 20 ft. if it is seriously intended and expected to avoid intermediate cracks. Such cracking as may appear within 20 ft. lengths would be relatively water tight and harmless except from the standpoint of appearance. Where water tightness is not a consideration, a clean, open joint as much as an inch in width is recommended. Reinforcement must be discontinuous whatever the type of joint. (Note difference of opinion—see foregoing discussion —EDITOR)

The Problem of Shrinkage in Masonry Units (39-110)‡

Q—It is recommended that concrete masonry units be dry when laid, as otherwise their shrinkage upon drying is likely to induce vertical cracks. However, during hot, dry, windy weather, it is difficult to obtain satisfactory mortar joints if the units are laid in a dry condition. Thus we seem to be confronted with a dilemma. What is the best solution?"

BY W. G. KAISER**

The question seems to me very much like the one that was put to the fellow who was asked if he had stopped beating his wife. He had to admit the premise was true before he could answer the question. Now as to the question before us. I can conceive that there may be times when, after prolonged spells of hot, dry, windy weather, the block may be dried out to such an extent that they may absorb an excessive amount of water from the mortar and therefore result in an inferior bond. Often the very opposite condition obtains, namely, that it is difficult to reduce and maintain the moisture content in the block below 40 per cent of the total maximum absorption as required in Federal, ASTM and other specifications.

*William T. Wright, "Experience with Weakened-Plane Joints," *Architectural Concrete* Vol. 5, No. 2, 1939, pp. 7-10; Portland Cement Association, Chicago.
†Bureau of Reclamation, Denver.
‡From 1942 ACI Convention "Quiz Session."
**Manager Cement Products Bureau, Portland Cement Assn., Chicago.

Assume then, that the block are too dry to obtain satisfactory bond. I would suggest that the following steps be taken to correct this condition. I would protect the block against drying out in the piles by covering or storing them in a shady place where they will not be exposed to the sun and wind. I would attempt to cover the walls as rapidly as the block were laid. I would use a somewhat more plastic consistency in the mortar so that the extraction of some water would not be detrimental. Furthermore, I would not stretch out mortar for bed joints for five or six units at a time but just enough for one unit and I would place that unit quickly before the mortar had a chance to thicken appreciably. If these precautions did not solve the problem, I would moisten the surfaces on the block which will come in contact with the mortar, doing it with a wet paint brush or whitewash brush. I believe this would slow up absorption sufficiently to make it possible to obtain satisfactory bond. I believe that Mr. Anderegg who has given a great deal of study to the subject of mortar will bear me out on this recommendation.

(At this point J. C. Pearson asked, "What order of unit shrinkage is permissible in concrete blocks to give reasonable expectancy that long exterior block walls will not crack across the blocks?")

Cracking in a masonry wall may be due to causes other than shrinkage and doubtless often is incorrectly attributed to shrinkage. Even when shrinkage is the primary cause of cracking, there are so many factors to be taken into account that it is very difficult to state definitely what is the optimum moisture content of the units to eliminate such shrinkage. In the first place, shrinkage in the wall may be accumulative and therefore, would depend upon the length of the wall. Shrinkage or volume change in a wall varies not only with the outdoor and indoor temperatures but also with the relative humidity.

The design of the building may be such as to impose restraints on certain sections of the wall. In other words, there are so many factors to be taken into account that Mr. Pearson's question is not directly answerable. In the ASTM and Federal specifications it is required that the block not contain more than 40 per cent of their maximum absorption when delivered to the job site. Block meeting this moisture content requirement may shrink when placed in the wall under certain conditions—particularly in buildings which are heated in the winter and where relative humidity is low.

Keeping these factors in mind I think we might answer the question about as follows: I believe that the chances of cracking in a masonry wall can be reduced to a minimum first, if the block meet the specifications in regard to moisture content; second, if the block are laid in the proper manner and the walls are not too long; third, if the building is properly

designed to provide contraction joints in long walls.

(On request Mr. Pearson then discussed his own question.)

BY J. C. PEARSON*

I cannot answer the question, and very likely no definite answer is possible. However, one might venture a guess that a shrinkage of 0.01 per cent would not produce cracks in most walls, whereas a shrinkage of 0.05 per cent might do so. We have given considerable thought to the possibility of finding some building involving a long block wall about to be erected, and arranging to have the blocks laid in damp condition. As soon as possible after the blocks were laid, a horizontal line of gage points might be set at convenient height, and the movements could then be followed for some time as the wall dried out. I think we need information of this kind.

One might speculate to some extent on the factors involved. It seems to me that the three most important factors are (1) the properties of the mortar; (2) the properties of the blocks; (3) the temperatures prevailing at the time of erection and subsequently. Data on the properties of the mortar and the blocks could be obtained in the laboratory, while measurements on the built up wall would give information on the effects of restraint, weather changes and other influences. An important objective in this sort of study would be the spotting of localized high stresses and the development of incipient cracks. Just how much the analysis of the measurements would reveal is impossible to predict with certainty, but it would help to answer the question that has been asked, at least for that particular wall.

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Current Reviews

of Significant Contributions in Foreign and Domestic Publications, prepared by the Institute's Reviewers

Wall and column footings without steel

VICTOR H. BERGMAN, *Engineering News-Record*, V. 129, No. 3, July 16, 1942.
Reviewed by S. J. CHAMBERLIN

Wall and column footings are designed as flat cantilever beams as allowed by the New York and Joint Committee codes. Diagrams facilitate the design.

Paving runways at half a mile a day clip

Engineering News-Record, V. 129, No. 1, July 2, 1942, pp. 70-71. Reviewed by S. J. CHAMBERLIN

With a single battery of equipment, 2200 ft. of 25-ft. wide concrete runways were paved day after day. The base course was placed by two pavers operating side by side on the subgrade. One paver operating outside the forms placed the top course after the reinforcing mat was installed.

Air-raid shelter for fifty people

Engineering News-Record, V. 128, No. 23, June 4, 1942, pp. 64-65. Reviewed by S. J. CHAMBERLIN

The underground, reinforced-concrete structure consists of a central room, 12 by 25 ft. with two entrances and passageways. The main walls and roof are 24 in. thick with heavy, double reinforcing. The floor is a 12-in. slab with ½-in. bars on 12-in. centers each way. Complete plans call for a 6-ft. minimum covering of earth and a 3-ft. thick burster slab.

Concrete pumped downhill for a dam

CHARLES M. GLINES, *Engineering News-Record*, V. 128, No. 21, May 21, 1942, pp. 83-85
Reviewed by S. J. CHAMBERLIN

An 8-in. pipeline was used to distribute 40,000 yd. of concrete to widely separated points for control works. Concrete with four and one-half bags of cement to the yard and at 1½ to 3-in. slump was dropped 70 ft. in the pipeline and pumped as much as 950 ft. Air vents tapped into the high point of the pipeline were opened at the beginning of a pour and closed when the pipe was filled with concrete.

Fast foundations in frozen ground

Engineering News-Record, V. 128, No. 21, May 21, 1942, p. 73. Reviewed by S. J. CHAMBERLIN

Coke was deposited in a strip about 18 in. wider than the desired trench. 24 hrs. after firing the ground was completely thawed and excavated with a trench hoe. Tents consisting of triangular timber frames covered with tarpaulins were placed behind

the excavator and heated with salamanders for hand trimming and forming. Truck mixers delivered the concrete containing heated aggregates and water. Tents were left in place for 14 days after concrete placing.

Saving reinforcing steel by rationalizing safety factors

A. J. Boase, *Engineering News-Record*, V. 128, No. 19 (May 7, 1942), pp. 81-83.
Reviewed by S. J. Chamberlin

The advantages of higher design units stresses are secured by the indirect procedure of using a reduced dead load for design purposes. In effect the safety factor for the fixed and calculable dead load is reduced, leaving the live load factor unchanged. A more balanced design between carrying capacities of slab beams and girders is obtained. The method requires no changes in building codes nor revision of handbook design data. A table compares the proposed design procedure (using partial dead load) with present design practice.

Concrete frames designed to save steel

Jaro J. Polivka, *Engineering News-Record*, V. 128, No. 25, June 18, 1942, pp. 83-86.
Reviewed by S. J. Chamberlin

Total steel demands and costs are compared for three examples of slabs designed for a live load of 100 lb. per sq. ft. and supported by columns 20 ft. apart in both directions. Conclusions are: (1) by using fillers, which cause no great increase in the dead load, steel savings up to 47 per cent are possible with one-way slabs with intermediate beams, and up to 56 per cent with two-way slabs; and* (2) using two-way slabs without a filler the steel savings may be as high as 46 per cent and at only slight variation in the total costs.

Wire saves steel in concrete joists

Engineering News-Record, V. 128, No. 23, June 4, 1942, pp. 90-92. Reviewed by S. J. Chamberlin

The floor system designed for housing projects uses precast joists with a 3-in. slab reinforced with standard wire mesh. Total steel requirements are claimed to be reduced by two-thirds. Wire stirrups, bundle wires for main reinforcement, and a single longitudinal stiffner wire are assembled in a jig for the joists. After the joists are delivered and set in place on 3-ft. centers, 2x3 in. wood purlins are rested on the ledges in the sides of the joists. 1x6 boards, three to the panel, are laid on the purlins as a base for the plywood panels that serve as bottom forms for the concrete floor. When the concrete is placed floor slab and joists are bonded together. The smooth ceiling and rough sand finish of the joists are painted. Costs were below wood construction.

New type pavement from mine tailings

Engineering News-Record, V. 128, No 25, June 18, 1942, pp. 87-90. Reviewed by S. J. Chamberlin

Lead and zinc mine "chats" are mixed in place with 12 per cent by volume of portland cement and 5 to 15 per cent of silty soil to produce a relative cheap pavement 6 in. thick. Chats and topsoil when distributed by slow moving dump trucks onto the prepared subgrade. Bagged cement was spotted and then spread by hand. The materials were mixed by rotary tillers. Additional water was added with pressure distributors as the mixing continued. Compaction was by sheepsfoot roller followed by a spring tooth harrow. Final operations were striking, blading, rolling with a pneumatic roller, followed by a 10-ton tandem smooth roller. The slab was covered with paper for seven days. It is planned to add a bituminous top coat later.

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*See ACI JOURNAL, Apr. 1942, Discussion by Thor Germundsson, "Saving Steel in Reinforced Concrete Design".

Studies of steel economy in concrete slabs and columns*

A. J. Boase, *Engineering News-Record*, Vol. 128, No. 21, May 21, 1942, pp. 86-87.
Reviewed by S. J. Chamberlin

Designs of seven types of concrete floor systems including supporting beams with three different live loads and three sizes of square panels indicate that the steel factor is the smallest for the flat slab and largest in floors with masonry fillers and in the two-way slab. Floors with metal pans and the one-way slab are between the extremes. In addition, neither flat slab nor solid slab ceilings need metal lath or plaster. The author's conclusions from studies related to reinforcement in columns are: the ACI code, 1936, is the most unfavorable; the New York and Chicago codes require less steel than the 1940 Joint Committee; where large loads must be carried by small columns, much steel may be saved by using high strength concrete; and, in general, tied columns require less steel than spiral columns.

Substantial service roads speed army base construction

Engineering News-Record, V. 128, No. 19 (May 7, 1942), pp. 64-67 Reviewed by S. J. Chamberlin

Early construction of the concrete roads that will ultimately serve the large warehouses and provision of temporary wooden surfacing units for construction equipment greatly reduced delays during the winter months. The construction of the concrete floors in advance of wall construction gave large open floors on which forms could be constructed and wooden roof trusses assembled. Exterior wall forms are set in two lifts for the full 24-ft. height as all concrete is placed from inside the warehouse. Truck-mounted mixers equipped with towers and counter-weighted chutes were used for placing concrete in the walls. Workmen doing the tamping and operating the vibrators work from platforms carried on brackets bolted to the inside forms. Concrete is placed in 8-ft. lifts; a 4-ft. depth being placed on the first run in any section of the wall and the remaining 4 ft. on the return run.

Proceedings Highway Research Board

Vol. 21, approx. 600 pages, $3.25

This is a record of the Highway Research Board's 21st Annual Meeting, 1941. The contents include contributions on finance, economics, design (including of special interest in the field of concrete—"Investigational Concrete Pavements"—Oregon, G. S. Paxson; Missouri, F. V. Reagel. "Effects of Joints in Concrete Pavements," F. V. Reagel and Don E. Gotham); materials and construction (including "Temperature and Moisture Variations in Concrete Pavement," F. C. Lang, "Vibration in Concrete Placement," H. F. Clemmer, "Measurement of Vibration Amplitudes," F. B. Hornibrook, "Soundness Tests of Aggregates," W. E. Gibson, "Effects of Calcium Chloride on Portland Cements and Concrete," J. C. Yates); maintenance, special papers (on roadside development); project committee reports and appendices.

Theory and measurement of plasticity and workability of mortars

Paul S. Roller, Forty-fifth Annual Meeting, American Society for Testing Materials, June, 1942.
Synopsis by Author

The plasticity of mortars is measured on the basis of a stress-strain relation that previously had been formulated and applied to various dispersions. Two constants are determined, coefficient of plasticity, π, and yield value, p_0, only the latter depending on the mixing water content.

Plasticity is defined in terms of ease of deformation and interpreted as one of the factors of workability of which four are discussed. π is shown to determine plasticity, and to be a good measure of over-all workability.

Water retention, as one of the factors of workability, was measured directly and with p_0 at 16.7 percent mixing water as an index. It was found to be well correlated with π for a series of 1:3 mortars of portland cement with added mineral powders. Relative to these materials at the same plasticity, the water retention of 1:3 mortars of several masonry cements and a slag cement seemed to be low.

Theoretical factors involved in plasticity and workability are discussed.

Curing of masonry mortars

HOWARD R. STALEY, Forty-fifth Annual Meeting, American Society for Testing Materials, June, 1942.
SYNOPSIS BY AUTHOR

This paper reports the effect on strength of 2-in. mortar cubes caused by variation in curing conditions. Fourteen variations in curing methods were used for lime-cement mortars and four were used for straight lime mortars. These variations in treatments include the effect of removal from the molds at 24 hr. and 48 hr.

A simple, economical storage has been devised using a converted ice refrigerator which yields results that are as consistent as obtained by using any of the other commonly used types of storage. The curing conditions obtained by using the converted ice refrigerator seem to be more suitable for all of the proportions of lime-cement mortars investigated than those obtained by any of the other storages used.

The storage most suitable for lime mortar specimens appears to be exposure to the laboratory air.

Since the lime-cement-mortar proportions investigated are considered to represent the full range of characteristics that would be obtained in masonry mortars, there appears to be no need for water storage of masonry mortars at any time during the curing period.

General features of Friant Dam

R. B. WILLIAMS, *Civil Engineering*, V. 12, No. 2, Feb., 1942, pp. 81-83. Reviewed by J. R. SHANK

Friant Dam is in the foothills of the High Sierras on the San Joaquin River about 20 miles north of Fresno, California. It is an important link in California's Central Valley Project. It is of concrete gravity section, center overflow spillway type, with a maximum height of 320 ft., a crest length of 3430 ft., a maximum base width of 268 ft., and a top width of 20 ft. It will contain approximately 2,150,000 cu. yd. of concrete. The first concrete was placed July 29, 1940. Special features are absorptive form linings on all exposed concrete faces, the addition of pumicite to all concrete except that for spillway faces, and limitations on concrete temperatures at the mixing plant.

Concrete aggregates were obtained three miles down river from the dam site and hauled by diesel-electric trains to track hoppers near the base of the dam, thence by belt conveyors to the mixing plant. The mixing plant consisted of four 4-cu. yd. mixers which discharge directly through a central cone into the 4-cu. yd. buckets of the diesel-electric concrete train which moved out onto the placing trestle where the buckets were picked up by hammerhead and whirley cranes working from the trestle.

Concreting a 100-acre office building

Engineering News-Record, V. 128, No. 23, June 4, 1942, pp. 80-84. Reviewed by S. J. CHAMBERLIN

450,000 cu. yd. of concrete are required for the War Department Office Building which will provide about 4,000,000 sq. ft. of gross office space and additional storage area. The plant batches material for 3,000 cu. yd. of concrete daily, feeding directly into batch trucks of 4 cu. yd. capacity for mixing enroute. Aggregate and sand are from the Potomac River and are delivered directly to the plant in barges in which they are originally loaded. The aggregates are unloaded by stiff-leg derricks and stock-

piled by a radial stacker. Belt conveyors carry aggregates to tops of batcher bins while bulk cement, delivered by rail in hopper bottom cars, is carried to a separate bin by bucket elevator. Moisture determinations are made by a specially built displacement apparatus. Compensation for moisture is made automatically on the special batching scale equipped with a compensating type poise. Most of the concrete for the structure is raised in 1 to 1½-cu. yd. tower hoists that discharge into suspended chutes leading to a hopper for loading the buggies. Column and slab forms are pre-assembled and marked for definite locations to facilitate re-use. Forms for the concrete walls to be left exposed in the inner court and light courts are rough on the face with carefully rounded edges to produce projecting fins.

Effect of brick absorption characteristics upon mortar properties

F. O. Anderegg, Forty-fifth Annual Meeting of the American Society for Testing Materials, June, 1942. Synopsis by Author

Absorption characteristics have been determined on several types of brick taken from different parts of the kiln. The abstraction of moisture by these bricks from lime mortar after contact lasting 1, 10, and 100 min. has been measured, and the setting times have been taken. The moisture gradients in lime mortars in contact with these bricks were also observed, and the effect of high initial absorption on mortar properties caused by the congealing of the mortar surface are described. Bond and joint-mortar compressive strengths, obtained with a 1:2:9 mortar at 28 days in contact with several types of brick, are plotted against 1-min. absorptions. The result on mortar strengths of wetting 3 of the more highly absorbent bricks is also shown graphically. Some observations on the effect of brick absorption on mortar durability are given. Strengths are recorded for joint mortars varying in composition from straight lime to straight portland cement for ages up to 1 yr. Their ratios to metal molded cube strengths are also given permitting one to make an estimate of mortar strengths in the wall. The problem of mortar durability is discussed in light of the British rule for choice of a mortar, and the need for more study of the deformability of masonry assemblages is emphasized.

Long-time volume changes of portland cement bars

Alfred H. White and Harold S. Kemp, Forty-fifth Annual Meeting, American Society for Testing Materials, June, 1942. Synopsis by Authors

This paper presents a continuation of the studies reported in the A.S.T.M. *Proceedings* for 1911, 1914, and 1928. There are available for study over 450 bars of known history which have been observed for periods ranging from 17 to 41 yr. The cements include a variety of commercial cements, and cements made in the laboratory whose composition differs in some cases quite widely from those found in commerce. A selection of 185 bars was made nearly 2 yr. ago for a special further study which involved alternations of the bars between the wet and dry states, with autoclave tests for some of the bars. Special attention was paid to the group carrying abnormally high magnesia.

All the cements retain their property of expanding when placed in water and contracting when placed in air. Progressive elongation continues when the periods of immersion in water are longer than those of dryness. Mortar bars expand and contract less than those of neat cement, but show larger volume changes than would have been expected from the amount of cement they contain. The chemical composition of the cement and the percentage of mixing water influence the extent of the volume changes somewhat, but do not change their nature. The autoclave test did not cause excessive expansion of these old bars except where they contained larger amounts of magnesia than are permitted by the standard specifications. Not all of the bars con-

taining high magnesia expanded in the autoclave test, since it apparently detects only free magnesia and not magnesia in combination.

Powdered aluminum as an admixture to concrete*

ROY W. CARLSON, Forty-fifth Annual Meeting, American Society for Testing Materials, June, 1942.
SYNOPSIS BY AUTHOR

When aluminum powder is added to a fresh concrete mix, the reaction of the aluminum and the mixing water releases hydrogen gas. If a substantial amount of the aluminum is used, the density of the concrete is reduced greatly and the strength is reduced accordingly. But if only around 0.01 per cent of aluminum is used, the effect is mainly to create a little internal expansion of the cement paste during hardening and to remove the usual "water gain" under aggregate particles and reinforcing bars. The net result is that compressive strength is not greatly affected, but some properties, especially bond to reinforcing steel are improved. Furthermore, when the amount of aluminum powder is such as to reduce the weight of the concrete seven per cent, greatly increased resistance to freezing and thawing (in $CaCl_2$) is found, without serious decrease in strength.

The paper shows the effect of amount, fineness, and method of adding aluminum powder, upon the properties of concrete. The paper covers tests on aluminum interground with a number of different clinkers, as well as simply admixed to the concrete.

The principal results of adding aluminum powder are (1) some decrease in strength for most concretes, but increased strength for harsh and lean mixes, (2) improved bond to reinforcing steel, and (3) improved resistance to freezing and thawing with calcium chloride.

What the citizen should know about civilian defense

WALTER D. BINGER and HILTON H. RAILEY, W. W. Norton & Co. Reviewed by W. G. BERGMAN

This is a book for the citizen and not for the person who wishes to become an expert in this new field for expertness—Civilian Defense. Yet the book does contain some things which this reviewer has not seen in the most scholarly and more technical books which he has read. For instance, there is a chapter on germ warfare which presents without hysteria the considerable possibility that an enemy in the realization of impending defeat might resort to the use of germs. The authors give some information from apparently reliable sources, indicating that Japan has already experimented with germ warfare in China on at least two occasions a year apart.

The book contains a very good summary of British experience with various types of shelters described. Probably the most important part of this discussion is a frank statement that we have to choose between using our resources for offensive effort and using them in defensive efforts such as construction of labor-consuming and material-consuming shelters.

Another valuable part of the book is a chart showing the safest place for refuge in different types of buildings. These recommendations are basically sound both from an engineering standpoint and from the equally important ground of common sense.

If you have plenty of time to become an authority on civilian defense, you probably would do better to read some of the books more elaborately equipped with tables and formulas than this one. However, if you feel that you can give only one evening to the subject, this reviewer knows of no better way to spend that one evening than by reading "What the Citizen Should Know about Civilian Defense."

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*See ACI JOURNAL, Nov. 1941, p. 183, 184; Jan. 1942, p. 274 (JPP Section).

Evolution of tremie-placed concrete dry docks

F. R. Harris, *Civil Engineering*, V. 12, No. 6, June, 1942, pp. 309-312. Reviewed by J. R. Shank

The immovable dry dock, known as the graving dock, embodies many very difficult problems of construction many of which are aggravated by the time factor. The author outlines some of the earlier procedures and finally stresses a procedure in which large amounts of tremie concrete are placed, though he states that the method outlined is no general panacea for constructing graving docks.

Following is a short statement of procedure used for constructing a dock at Philadelphia. After the bottom was prepared with a two-foot thick bed of gravel, steel H piles were driven below subgrade. Steel tremie concrete floor forms extending for the full width of the structure were placed on the gravel bed and the steel truss reinforced floor was tremie poured to the inside faces of the side walls. After this concrete had set sufficiently the wall forms were set on the floor forms whereupon the walls were tremie poured from subgrade to the tops of the forms. Prior to this, however, a line of sheet piling was suspended from a continuous horizontal waler supported above the top of the wall form. Upon completion of the tremie walls, backfill was placed, and after this the dock was unwatered for completion in the dry. This dock was constructed in less than two years.

It is still possible that a faster and more economic procedure may in some cases be full cofferdaming by some means and unwatering the site.

The author discusses generally the hydraulic and hydrostatic conditions that the designers must take into account.

Saving steel by substituting concrete

Hardy Cross, *Civil Engineering*, V. 12, No. 7, July, 1942, pp. 365-367. Reviewed by J. R. Shank

Professor Cross calls attention to some things to consider when designing to conserve steel. He quotes A. J. Boase and T. D. Mylrea.

Timber, plain concrete, brick, or clay pipe should be used if possible; otherwise combinations of familiar type—one material for tension, another for compression—reinforced concrete. Use reinforced concrete for bins, bunkers and tanks, where straps at high stresses replace plates. Note that the amount of tensile steel needed for flexure is less if there is also direct compression in the member-arches and dome construction. Concrete roof spans up to 150 ft. are feasible; I-sections at the ends and Vierendeel types in the mid-portions are suggested. Continuous and over-hanging beams and frames save material. Design in three dimensions—domes and shells. Use walls as beams and single members to resist forces acting in different planes.

Codes and specifications should be revised. A working stress of 30,000 psi (for 60,000 psi elastic limit) is not too great provided secondary failure due to bond or anchorage is guarded against. Simple beams where anchorage and shear are properly cared for show in tests that all cracks are very fine.

Use thin slabs and reconsider deflection requirements. Flat slab decks are estimated to give minimum steel, the advantage increasing with the live load.

Where savings have been tried, contractors have proven themselves unusually versatile in switching from steel to concrete and surprisingly small amounts of difficulty in the field have been encountered. Engineers should look to their own inertia.

Concrete mixing plant for Friant Dam

C. T. Douglas, *Civil Engineering*, V. 12, No. 4, Apr., 1942, pp. 191-193. Reviewed by J. R. Shank

The concrete mixing plant was along the San Joaquin River about 200 ft. down stream and 100 ft. lower than the crest elevation. Sand and four sizes of gravel were stored 970 ft. down stream from, and 250 ft. below the mixing plant. The mixing

plant had three levels. Materials entering at the top level were fed by gravity to the second on batching level and then to the third or mixing level. Four 4-cu. yd. mixers clustered around a central feed. The plant was provided with individual batchers for sand, gravel, cement, pumicite, water and tempering water. Each batcher had its individual scales, cut off, and weighing beams and a full-capacity dial. All batcher charging was controlled electrically using air operated gates or valves from a central point. The tempering water was a slush ice bearing water developed at a refrigerating plant which was used for bringing the temperature of the batch down to the 70 F. temperature specified.

The maximum mixing efficiency was attained in the minimum time when the mixers were charged by threading the materials into the mixers according to a sequence. Cobbles and the start of the water introduction were first. The next stage consisted of sand, fine gravel, intermediate gravel and pumicite, followed by the cement and coarse gravel. The water continued through and finished up the sequence. Approximately 15 seconds were required to charge the mixer. The total time to tilt, discharge and return the mixer to the charging position was 12 seconds. The mixing time was from 2½ to 2 minutes.

All of the mixers were provided with consistency indicators actuated by the shift of position of the batch in the mixer as the mixing progressed. Each batch mixing was automatically timed by releasing a locking mechanism and all operations were automatically recorded. The recorder indicated times of all operations, consistencies, the drops of the weighing beams and all weights.

The average production was 6000 cu. yd. per day with a maximum recorded of 9059 cu. yd.

Wide-span hangers for the U. S. Navy

ANTON TEDESKO, *Civil Engineering*, V. 11, No. 12, Dec., 1941, pp. 697-700. Reviewed by J. R. SHANK

A general description and discussion of the structural features and design problems for a twin hanger of arch-type are given with few or no dimensions. The roof structure is of Z-D design. The arch ribs are outside leaving a plain curved interior. One set of centering and forms was used and shifted from unit to unit. One unit consists of two bays, a full bay between two ribs and two half-bays. The hinges were of the Mesneger type. The horizontal thrust of 39 tons is taken by prestressed used bridge cables, eight $1\frac{1}{16}$-in. strands. As the arch centering was lowered the 7½ in. stretch of these cables was taken up at the footing structures. The Mesneger hinges were 80 in. long with 32 1¼-in. round rods vertically and 20 1-in. round rods horizontally in a fan-like arrangement. The end arch ribs were special to take the torsional effects produced by the doors. Each part is designed for separate stability and the roof is designed so that bombing will produce the least amount of general damage to the structure as a whole.

The concrete mix containing 6½ bags of cement and for 3500 psi was mixed to a 3½ in. slump and was pumped through 8-in. pipes to the crown at a point midway between ribs where it branched and extended both ways for about 80 ft. From here it was distributed by elephant-trunk chutes. The concrete of each roof unit was poured in a continuous operation for 15 hours. When the concrete had attained a strength of 2500 psi and a modulus of elasticity of 2,500,000 psi decentering was started gradually while the deflections of the arch, the stretch of the tension ties and the movements of the footings were carefully controlled. Decentering took place from 5 to 7 days after pouring. A main roof pour was made every 2½ to 3 weeks. Expansion joints between roof units were covered by sliding copper flashing.

Experience with the "Adherit" concrete road

K. KRAH: *Verkehrstechnik*, 1941, 22 (4), 57-8. Road Abstracts, Vol. IX, No. 3, p. 27, March, 1942. HIGHWAY RESEARCH ABSTRACTS

The "Adherit" surfacing has been in use in Germany since 1932 with satisfactory results. The method may be used on badly worn macadam or sett paving, or in new construction. Old surfacings are cleaned, spread with tar at the rate of 1.84 lb. per sq. yd., and at once covered with stone chipping (0.32 to 0.6 in.). A leveling course of concrete is spread to a minimum depth of 1.2 in. This obviates any leveling or reshaping of the original surfacing. Stone chippings (0.5 to 1 in.) are rolled in at the rate of 46 lb. per sq. yd. A bitumen or tar emulsion is then applied at the rate of 1.84 lb. of bituminous material per sq. yd. Certain criticisms and recent developments are discussed. The 1.2-in. (3-cm.) depth of concrete, which is sometimes regarded as insufficient, is a minimum; the depth is in places considerably greater if the original surfacing is uneven. The 1.2-in. concrete course is not intended to be a bearing course, the functions of which are fulfilled either by the existing surfacing or, in new construction, by the lower-course concrete. To ensure good bond between the surface chippings and the concrete it is necessary to take care of the emulsion water. To this end the concrete has been tried with only half its proper proportion of water, and it is now found that the concrete is best laid without any mixing water. Pit-moist sand or gravel is mixed with the cement and laid dry. The emulsion is diluted with water to provide the requisite amount for the dry concrete, which absorbs the water when the emulsion is applied. The method of construction prevents drying-out of the concrete and obviates shrinkage, while the low thermal conductivity of moist concrete results in little volume change due to temperature. "Adherit" surfacings are given a single or double surface dressing, either immediately or later. If traffic is not heavy the second dressing may be given in the second year. In new construction, a sand insulation course is laid on the formation and covered with bituminized paper on which ordinary concrete is spread to a depth of 2 to 2.4 in. An upper course of dry concrete is at once applied and the surfacing is completed as already described.

Grading of gravel aggregates and quality of concrete

H. N. WALSH. Reprint published by the Institution of Civil Engineers of Ireland, 1942, Proc., pages 177-224. Reviewed by J. C. PEARSON

This paper reports a continuation of the investigations by the author, which have extended over a period of several years and are designed to simplify the methods of proportioning concrete for contractors and engineers who do not have elaborate testing facilities at their disposal. The selection of proper proportions is based on certain sieve analysis curves designated by the author as "type gradings" and established by him for Cork gravel. These type gradings are indicated for a range of proportions of cement to gravel, and show, as would be expected, that finer gradings are required for leaner mixes. From these basic curves for any given size of maximum aggregate other curves are derived, such for example, as those which show the variations in cement content and water content for various values of n (cubic feet of aggregate per sack of cement).

Departure from the type-gradings, of course, affects the quality of the concrete, and it is mainly the effect of these (finer) gradings that the author brings out in the present paper, and shows by means of relatively simple diagrams what quality of concrete can be expected when the grading of any given material has been established.

The success of Professor Walsh's method seems to depend on the selection of a ratio between given fine and coarse aggregates that brings them into closest conformity with the type-gradings. If the optimum grading thus established does not have a definite trend with the type-gradings, and wanders more or less across or outside of

the prescribed zones, then the designer would be somewhat at sea. The same thing might occur if aggregates of decidedly different shape from the Cork gravel were to be used. However, barring the exceptional cases, the author declares that cement contents of trial mixes with unfamiliar aggregates generally agree closely with expected values, mostly within 2 per cent. It is to be borne in mind that scientific accuracy is not expected in the results obtained, but that the method is a simple and practical one in the absence of information on such things as specific gravity, absorption, absolute volume, etc.

Concrete production and control for Saugatuck Dam

W. R. Miller, *Civil Engineering*, V. 12, No. 2, Feb., 1942, pp. 88-90. Reviewed by J. R. Shank

The Saugatuck Dam is on the Saugatuck River about 12 miles northwest of Bridgeport, Conn. It has a storage capacity of 12 billion gallons and covers 870 acres. The main dam is 1000 ft. long at the top, 125 ft. high and 100 ft. thick at the base. It contains 80,000 cu. yd. of concrete.

Aggregate used comes from a glacial deposit about three miles upstream from the dam. It is washed and the oversize pieces are crushed. It is screened into sand at 3.00 fineness modulus and three sizes of gravel, the largest being 6-in. Bulk cement was used.

The concrete mixes were, for exposed faces, 1.06 bbl. of cement per cu. yd. with a water-cement ratio by weight of 0.54 and a slump of ½ in., and 0.93 bbl. with a water-cement ratio of 0.61 and slump of ½ in. for core concrete. These two produced strengths of 4200 and 3600 psi respectively at 28 days and 5710 and 4920 psi respectively at one year.

Strict control and records of the mixing were kept. At least four times every hour although at times for every batch, samples of the sand and coarse aggregates were tested for moisture by means of a moisture meter scale which, without calculation, determines the percentage of free moisture to within one-quarter of one per cent. A moisture test requires not more than one minute. Compensation for the moisture was made automatically on the batching scale which was a full dial scale having a separate beam and poise for each ingredient, including water. Each aggregate that carried free moisture was assigned to a special scale beam equipped with a compensating-type poise so designed that by adding small disks, calibrated in per cent, its weight can be increased by the desired percentage. Thus when a poise is set to the weight of dry material required, and disks totaling the percentage of surface moisture carried by the material are added, the dry weight plus the weight of the contained moisture is shown on the dial. After each aggregate has been weighed in this manner, the poise assigned to water is set at the total weight of the water required for the batch and the percentage disks previously added are removed. This automatically deducts the amount of water carried by the aggregates from the total required for the batch, leaving only the net amount to be added.

Aggregate production for Friant Dam

C. T. Douglas, *Civil Engineering*, V. 12, No. 3, Mar., 1942, pp. 127-130. Reviewed by J. R. Shank

The aggregates for the Friant Dam were obtained from a fluvial deposit three miles down the Joaquin River from the dam site. This deposit contained several types of gravel, some of igneous origin including andesite, syenite, basalt, and some sedimentary and metamorphic types among which were quartzite and other siliceous rocks. The finer portions of the sand contained appreciable quantities of weathered feldspathic minerals among which was mica to the extent of 4 per cent by volume. Only about 2 per cent was larger than 6-in. square screen.

Belt conveyors were used from the grizzly field hoppers to a 3500-ton live storage pit. From an underfeed at this raw storage pile a belt conveyor carried the material through an 8-in. scalping screen, which took out the larger material and by-passed it through a gyratory crusher to the screening plant. The coarse aggregate was separated into four sizes from 8-in. to $\frac{3}{16}$-in. The fine was separated into two sizes and recombined to a fineness modulus of 2.75. At the stock pile a reversible traveling shuttle conveyor permitted equal distribution of the wet sand over the entire storage, thereby making possible a uniform moisture content in the sand loaded out to the concrete mixing plant. Rock ladders were used to reduce breakage and segregation.

Conveyors carried the finished material from the bins to six 50 ton loading hoppers along a standard gauge track. Six standard hopper cars could be loaded simultaneously in 25 seconds.

The cement in three brands was delivered in bulk and unloaded at the rate of 400 bbl. per hour into an 8-in. pipe line which delivered it to four 5900 bbl. capacity silos.

Pumicite was obtained from a deposit three miles from the dam site. The finest of this, 95 per cent through a 325-mesh screen was added in the proportion of 20 per cent of the cement by weight to all the mass concrete except the six foot layer on the surface of the spillway section. Eight tenths of a barrel of cement and 60 pounds of pumicite were used in this mass concrete.

The sand contained gold to the amount of five cents per ton of pit run gravel. Chutes with burlap screen lining on the bottoms and other screens were used to concentrate this gold. The amalgamation process was used on these concentrates.

Construction plant and methods—Friant Dam

D. W. WALTER, *Civil Engineering*, V. 12, No. 5, May, 1942, pp. 259-262. Reviewed by J. R. SHANK

A steel trestle 2430 ft. long with two double hammerhead and two revolver or whirly cranes was the principal feature of the construction plant of the 3430-ft. long dam. This trestle is 129 ft. down stream from the axis of the dam, the deck of which is 217 ft. above the lowest bedrock foundation and 103 ft. below the parapet of the dam. It is supported on tower sections in alternate blocks of the dam. Recesses formed around the columns and bracing members, where they project through the downstream face of the dam were filled with concrete when the upper portion of the trestle was removed at the completion of the contract. Hatchways through the deck permitted buckets of concrete to be lowered through the trestle deck when concrete was being placed directly under the trestle. A small trestle was erected 116 ft. downstream from the main trestle for early stages of construction.

The foundation was treated by grouting through $1\frac{3}{8}$-in. holes. Low pressure shallow grouting was done through holes in three rows 25 to 30 ft. deep on approximately 20 ft. centers. The main cut-off or grout curtain consisted of deep holes on approximately 5 ft. centers on a line downstream from and parallel to the axis of the dam. Neat cement grout with water-cement ratio of 5.0 to 10.0 was used. When any hole accepted grout at a fair rate, this ratio was gradually reduced until the hole was accepting the maximum number of sacks of cement per hour, at the maximum allowable pumping pressure of 150 psi. The grout acceptance was relatively small; however one hole took more than 1600 sacks of cement. The shallow low pressure holes took 21,568 sacks in 725 holes with an average acceptance of 0.97 sack per foot of drilled hole.

The general concrete mix was 0.80 bbl. of low heat cement per cu. yd. with 20 per cent by weight of pumicite. At the downstream face of the spillway, 1.00 bbl. of cement was used. The blocks were 50 ft. wide generally and varied in length from 30 to 270 ft. The concrete, slump, $1\frac{1}{4}$ to $1\frac{3}{4}$ in. was vibrated into place in 12-in. layers placed at

intervals of not more than 1 hour, to 5 foot lifts. The minimum interval between 5 foot lifts was 72 hours. The lift tops were slightly concave with sumps at the center. Before starting a new lift the top face of the old concrete was wet-sand blasted and washed. Cantilever forms secured by bolts connecting 12x4 in. walers to nuts held by embedded hooked ½-in. diameter steel bars were used. An adjustable roller device acting on the face of the concrete was used when the 50 foot form sections were shifted. The minimum stripping time was 24 hours in summer and 36 hours at other times.

For cooling, river water was pumped through pipes embedded in the concrete. The moist curing time was four weeks for the general mix and three weeks where pumicite was not used.

Testing and inspection of engineering materials

DAVIS, TROXELL, AND WISKOCIL, McGraw-Hill Book Co., 372 pages, 7 x 10 in., lithoprint preliminary edition for engineering defense training classes. ($3.50)

Reviewed by GEORGE W. WASHA

This excellent book contains information for all that have any contact with testing and inspection of materials. It has been divided into two main sections, the first dealing with the principles of testing and inspection, and the second dealing with instructions for laboratory work.

The authors have devoted the first chapter to a discussion of the nature of the problem in which such items as selection of materials, inspection, properties, specifications, design of tests, and many others are well covered. The second chapter defines fundamental terms used, with explanatory comments.

The third chapter deals with load and deformation measurements and apparatus for making them. Good descriptions of most known load and strain measuring devices are accompanied by details of their principles of operation.

The next three chapters consider static tension, compression, shear, bending, and hardness tests. In each case a thorough discussion of test specimen, conduct of test, and observations during test are given, and the effects of important variables are pointed out. The chapter on hardness tests is featured by descriptions of the many hardness measuring devices accompanied by detailed descriptions of operation. This is followed by excellent chapters on impact tests, and on fatigue and creep tests of metals.

Another outstanding chapter in the book is that dealing with nondestructive tests. This particular field of testing has become increasingly important in recent years, and descriptions of the various methods are presented. Among the nondestructive tests described are: oil-whiting, stethoscope, drill, X- and Gamma-ray, magnetic analysis, magnaflux, proof, and Sperry detector.

The question of the proper analysis and presentation of data is next considered. The authors discuss the methods of handling variations in data and give formulas for calculating arithmetic mean, average deviation, standard deviation, coefficient of variation, and other important statistical values, and also discuss the use of control charts and scatter diagrams.

The second part of the book contains 20 experiments that might be used for classroom purposes to demonstrate the various tests, to acquaint students with the use of testing equipment, and to provide them with a knowledge of the more important properties of the commonly used structural materials. Among the suggested tests are the following on cement and concrete: "Compression Test of Concrete," "Flexure Test of a Reinforced Concrete Beam," "Characteristics of Fresh Concrete," and "Effect of Water-Cement Ratio and Cement Content Upon Quality and Economy of Concrete Mixtures."

The experiments are followed by brief summaries of the properties of the following: ferrous metals, wire rope, wood, concrete, brick and plastics. These summaries provide a large amount of condensed information for each material, but only the more important facts have been presented in each case because of the space limitations. More than 1000 references is found at the end of the book.

Relative merits of wire and bar reinforcement in pre-stressed concrete beams

RHYDWYN HARDING EVANS, M.Sc., Ph.D., Assoc. M. Inst. C. E. *Journal of the Institution of Civil Engineers*, No. 4, 1941-42, Feb., 1942, p. 315. (London). HIGHWAY RESEARCH ABSTRACTS

The principle of pre-stressing concrete members is to introduce initial stresses in the concrete of opposite sign to that produced by the dead and live loads, thereby eliminating tensile cracks in the concrete by maintaining a fully loaded beam with compression over the whole section. For this purpose M. Freyssinet has used both high-tensile steel bars and high-tensile steel wire or piano wire, the latter varying from 1/25 in. to 1/8 in. in diameter, with a tensile strength of from 150 tons to 175 tons per sq. in., and also steel bars with no bond between the steel and concrete, the bars thus acting as tie-rods. The permissible stress in the piano-wire tension reinforcement is approximately three times that in the high-tension bar reinforcement, so that the required cross-sectional area of wire is one-third of that of the bar reinforcement. The surface-area of the wire, on the other hand is $\sqrt{\frac{n}{3}}$ of that of the bar reinforcement for the same initial pre-stressing force, where n denotes the number of wires utilized as tension reinforcement. If wires of 0.08 in. diameter are used, the surface-area of the wire reinforcement is about four times that of the bar reinforcement, with a corresponding reduction in the bond stress for a given shear-force. These differences in the permissible stresses and dimensions of the wire and bar reinforcement, together with the possibility of eliminating the bond in bar beams, will influence the behavior of pre-stressed concrete beams when reinforced with those types of reinforcement. No investigation seems to have been made so far to determine the relative merits of wires and bars in pre-stressed beams, and in this paper some experimental results are presented concerning the deflection, hysteresis loops, permanent set, and cracking loads of such beams for increasing values of initial pre-stressing force. The effect of introducing tie-rods without any bond is also considered.

Conclusions. The experiments described were made in order to compare the relative merits of wire and bar reinforcement in pre-stressed concrete beams. The tests were carried out on beams of 10-ft. span, and a series was made with various values of initial pre-stressing force. The central deflection and the compressive and tensile strains in the concrete were recorded. Graphs are given to show that the permanent set in all pre-stressed concrete-beams is exceedingly small and is no more than that in plain concrete-beams. It is shown that the cracking load of pre-stressed beams with bar reinforcement is considerably lower than that of pre-stressed beams with wire reinforcement, being only about one-half for an initial pre-stressing force of 25 tons. This is an important difference between the behavior of bar and wire pre-stressed beams, as the working load should not exceed that necessary to produce tensile stresses in the concrete. The reason for this difference is explained by investigating theoretically the loss of initial pre-stressing force due to straining, creep, and shrinkage of the concrete. The percentage-loss in the bar-beams is approximately three times that in the wire-beams, the loss being a minimum when the steel stress is a maximum. The experiments show that piano-wire is a very efficient type of reinforcement in pre-stressed concrete, and that the Bernoulli-Euler theory has proved adequate in calculating the concrete fiber-stresses and the cracking-loads.

The behavior of pre-stressed concrete beams with no bonded tension reinforcement is also investigated experimentally and theoretically. It is shown that the stresses in the tension steel and the concrete, as well as the deflection, are considerably higher in no-bonded pre-stressed beams than in either the conventional type of concrete beam or the fully-bonded pre-stressed beam.

Rectangular footings—determination of Ultimate Bearing Pressure

The Surveyor and Municipal and County Engineer, Vol. CI, No. 2610, p. 38, Jan. 30, 1942, London, from H. Q. Golder, Journal of the Institution of Civil Engineers. Highway Research Abstracts

The author (of the Building Research Station) introduces the subject by pointing out that there are two ways in which failure of stability of a footing can take place, namely, "by excessive settlement of the footing due to consolidation or deformation of the underlying soil strata, or by shear failure of the soil." The allowable pressure will therefore be the smaller of the two values: (1) The pressure which causes settlements which do not exceed a certain specified value; (2) the pressure which gives a chosen factor of safety against shear failure. In the paper, only shear failure is considered, and the term "ultimate bearing pressure" refers to the pressure which causes shear failure. In regard to extrapolation from results of small-scale tests, the author points out that the influence of size and shape should be known. He observes that all methods of analysis indicate that the ultimate bearing pressure is directly proportional to the width of the footing on a purely frictional soil, and independent of the width of a purely cohesive soil.

Loading Tests on Sand. The author describes tests carried out with footings of various lengths, but of equal width, on sand, to decide the effect of the length of the footing upon the ultimate pressure. The sand was a clean, dry, river sand of $\frac{3}{16}$ in. and less; in percentages, less than: 1 mm., 86; 0.72 mm., 80; 0.5 mm., 60; 0.4 mm., 47; 0.3 mm., 30; 0.2 mm., 8.

A stout wooden box, 3 ft. 6 in. square and 14 in. deep, was filled with the sand which was well tamped with a 3-in. square steel plate attached to an electric hammer. From test to test the variation in weight per cu. ft. was from 108 to 112 lb., with a mean of 110.5 lb. corresponding to a voids ratio of 0.51.

Method of Test. The footings used were five 6-in. rolled steel channel sections, respectively 6, 12, 18, 24 and 30 in. long, placed on their backs and filled with concrete. The load was applied by a hydraulic jack through a proving ring by which the load was measured. The settlement was recorded by means of a vertical scale attached to the footing.

Ultimate Pressures on Sand. The width of the footing being 6 in. in every case, the ultimate pressures in 17 tests were, in tons per sq. ft., with the lengths, 6 in., 2.61, 2.45 and 2.86; 30 in., 2.46 and 2.46; 24 in., 2.42 and 2.68; 18 in., 2.41 and 2.58; 12 in., 2.73 and 2.67; 6 in. 2.25 and 2.47; 24 in. 2.53; 12 in., 2.44; 6 in., 2.91; 30 in., 2.93. The author observes that "it is clear that despite a certain scatter of the individual results, there is no tendency for the ultimate pressure to increase with length." It will be noted, also, that any change in the condition of the sand was cared for by the order of variation of length of footing.

During the test, and as shown in photographs accompanying the paper, the sand rose all round the footing, and the outline of the surface of failure was clearly marked on the sand.

The author discusses theoretical analysis in the light of these results. Figures include graphical relationships on: (1) Ultimate Pressure and Shape of Footing, (2) Ultimate Pressure and Width of Footing, (3) Bearing Pressure and Length of Footing,

(4) Variation in Shear Strength with Normal Load for Dense and Loose Sand, (5) Variation of Angle of Internal Friction with Normal Load for Dense and Loose Sand, and (6) Variation of Bearing Pressure with Angle of Friction.

Bearing Tests on Clay. The author observes that, theoretically, breadth has no influence upon the bearing pressure of a footing on a plastic material such as soft clay, the bearing pressure depending only upon the shear strength of the material. The theoretical bearing pressure is, however, calculated for an infinitely long strip footing, and it is desirable "to know the relationship between such a load and a square footing of equal width." To obtain some indication of that relationship, the following tests were carried out.

Ultimate Pressures on Clay. The width of the footing being 3 in. in every case, the ultimate pressures in six tests were, in lb. per sq. in., with the lengths, 3 in., 12.76; 18 in., 9.86; 3 in., 12.50; 18 in., 9.64; 18 in., 9.71; 3 in., 12.30; means on 3 in. by 3 in.; 12.5 lb. per sq. in., on 18 in. by 3 in., 9.7 lb. per sq. in. The author draws the conclusion "Therefore it would appear that for footings of this size on a plastic foundation material, the ultimate pressure on a square is approximately 30 per cent higher than on a long rectangle of equal width."

Observed and Calculated Bearing Pressures. From seven shear tests, the mean shear strength of the clay of mean water content 42.2 per cent was 1.88 lb. per sq. in. The mean bearing pressure for the long rectangle is 5.1 times that value comparing with L. Prandtl's 5.14; and with G. Wilson's 5.41, found by a mathematical solution of H. Krey's graphical method ("The Calculation of the Bearing Capacity of Footings on Clay"; Journal Inst. C. E., November, 1941). M. Ritter's formula gives 4.0.

The paper also contains a bibliography of eleven references.

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"National Emergency Specifications for the Design of Reinforced Concrete Buildings" to Conserve Steel

SYNOPSIS

To conserve reinforcing steel, as a war measure, Emergency Specifications issued by WPB, are the substance of Directive 9 which gives them force for all government war time reinforced concrete building construction. Here the new Emergency Code provisions are shown by comparison with ACI 318-41.

By the War Production Board's Directive No. 9 (signed by Chairman Donald M. Nelson, October 5, 1942) new National Emergency Specifications for the Design of Reinforced Concrete Buildings supersede the Institute's Building Regulations for Reinforced Concrete (ACI 318-41) in respect to provisions governing the use of reinforcing steel for all buildings constructed, financed, or approved by government agencies on contracts placed after December 4, 1942. Designs shall be changed immediately to agree with these Emergency Specifications unless such changes will result in a substantial delay in the war effort.

The purpose of the directive is to conserve the supply of reinforcing steel by requiring the use of larger structural members and higher tensile stresses than have normally prevailed. Allowable compressive stresses in concrete have been reduced and stresses for steel increased from 18,000 to 20,000 psi for structural grade bars and 20,000 to 24,000 psi for intermediate and hard grade bars. Other changes from prevailing peace-time practice include: increasing from 50 to 60 per cent of yield point the stress in the reinforcement of one-way slabs under 12 ft., retaining the present 30,000 psi maximum; increasing bond stresses; requiring plain concrete where practicable for footings, walls and piers, gravity and semi-gravity retaining walls and buttresses, and, to increase the practicability of such uses, raising the allowable tension in unreinforced concrete from 3 to 4 per cent of the 28-day compressive strength, f'_c; and to make the concrete do more work the permitted compressive stress in concrete is reduced from $0.45\,f'_c$ to $0.35\,f'_c$.

The Emergency Code was prepared under the general direction of C. L. Warwick, Chief of the Specifications Branch of War Production Board's Conservation Division, and under the immediate supervision of C. A. Willson, Chief of the Civil Engineering Section and Edwin Joyce, Chief of the National Emergency Steel Specifications Section. A group of engineers and designers from the governmental agencies and the construction industry collaborated in the development of these specifications. Prominent among these men were H. P. Bigler, Connors Steel Co., H. B. Zackrison, Corps of Engineers, War Department, and Commander G. A. Hunt, Bureau of Yards and Docks, Navy Department.

ACI members and thousands of other users of the "ACI Code" (ACI 318-41) in the United States and elsewhere, especially those who have contributed to or been interested in the ACI discussions of steel conservation,* will be interested to learn the specific savings of steel by departures from ACI regulations in promulgating "Emergency Specifications." Chapter 1 is therefore published here. It is superimposed upon Chapters 6 to 12 of the ACI Code, revised as will be indicated in these pages, to comprise Chapters 2 to 8 of the War Time code.

Copies of the new Emergency Specifications (Directive 9) may be had from the War Production Board† It is reviewed here with special reference to its departures from the ACI Code, because of the years of design practice on reinforced concrete buildings based largely on the current (ACI 318-41) and previous ACI Codes which have been widely accepted as the basis of reinforced concrete building design.

The Emergency Code does not deal with the subject matter of ACI Code Chapter 2, Materials and Tests, Chapter 4, Mixing and Placing Concrete, or with Chapter 5, Forms and Details of Construction. Emergency Code Chapter 1, General Requirements and Working Stresses, replaces ACI chapters 1, General, and 3, Concrete Quality and Working Stresses, and Chapters 2, 4 and 5 (already mentioned) and employs as its Chapters 2-8, ACI Chapters 6-12 respectively with the specific revisions to be noted.

*See R. L. Bertin, "Saving Steel in Reinforced Concrete Design," ACI JOURNAL, Feb. 1942; *Proceedings* V. 38, p. 281 and voluminous discussion by 16 contributors, April JOURNAL and Nov. *Supplement*, 1942, p. 288 - 1.

†P. H. Colby, 1310 Railroad Retirement Building, 3rd and C Streets, S.W., Washington, D. C.

NATIONAL EMERGENCY SPECIFICATIONS FOR THE DESIGN OF REINFORCED CONCRETE BUILDINGS

CHAPTER 1—GENERAL REQUIREMENTS AND WORKING STRESSES

100—Notation

f_c = Compressive unit stress in extreme fiber of concrete in flexure.
f'_c = Ultimate compressive strength of concrete at age of 28 days unless otherwise specified.
f_s = Tensile unit stress in longitudinal reinforcement; nominal working stress in vertical column reinforcement.
u = Bond stress per unit of surface area of bar.
v = Shearing unit stress
v_c = Shearing unit stress permitted on the concrete.

101—General requirements

(*a*) Reinforced concrete designs and details shall be so selected as to use a minimum amount of steel reinforcement. To accomplish this, the design shall embody a maximum of symmetry and simplicity of layout and a minimum of ornamentation and shall comply with the requirements of the Directive for War-Time Construction, dated May 20, 1942, and the "List of Prohibited Items for Construction Work" of the Army Navy Munitions Board, dated June 29, 1942, as amended. Non-reinforced concrete or masonry shall be used in footings, walls and piers of substructures, gravity or semi-gravity type retaining walls and buttresses in lieu of reinforced concrete construction wherever practicable. Fill under concrete slabs shall be thoroughly consolidated so that the reinforcement may be reduced to a minimum or eliminated entirely.

Wherever practicable, the width and depth of members shall be increased to avoid the use of compressive reinforcement and to minimize the use of web reinforcement and special anchorage. Wherever prestressed reinforced concrete construction can be used to advantage with resulting savings in steel, it shall be adopted.

The amount of reinforcement in concentrically-loaded columns shall be kept to a minimum by:

1. Using rodded columns in preference to spiral columns.
2. Using not less than 0.5 per cent and not more than 2.0 per cent of longitudinal reinforcement.
3. Using high strength concrete.

102—Allowable unit stresses in concrete

(*a*) The unit stresses in pounds per square inch on concrete to be used in the design shall not exceed the following values where f'_c equals the minimum specified ultimate compressive strength at 28 days, or at the earlier age at which the concrete may be expected to receive its full load.

TABLE 102(a)--ALLOWABLE UNIT STRESSES IN CONCRETE

Flexure f_c

Extreme fiber stress in compression—but not to exceed 900 psi ..... $f_c = 0.35 f'_c$
(Upon certification by the designer that an overall saving of steel will be effected, this allowable stress may be increased for special members or special details to a maximum of $0.45 f'_c$ but not to exceed 1200 psi.)

Extreme fiber stress in tension
Wherever plain concrete is used in flexure the tension in the extreme fiber shall be $0.04 f'_c$.

Shear v

Beams with no web reinforcement and without special anchorage of longitudinal steel ..... $v_c = 0.02 f'_c$
Beams with no web reinforcement but with special anchorage of longitudinal steel ..... $v_c = 0.03 f'_c$
Beams with properly designed web reinforcement but without special anchorage of longitudinal steel ..... $v = 0.06 f'_c$
Beams with properly designed web reinforcement and with special anchorage of longitudinal steel ..... $v = 0.12 f'_c$
*Flat slabs at distance d from edge of column capital or drop panel ..... $v_c = 0.03 f'_c$
**Footings ..... $v_c = 0.03 f'_c$

†Bond: u

In beams and slabs and one-way footings:
Plain bars ..... $u = 0.044 f'_c$ but not to exceed 160 psi
Deformed bars ..... $u = 0.055 f'_c$ but not to exceed 200 psi
In multiple-way footings:
Plain bars (hooked) ..... $u = 0.050 f'_c$ but not to exceed 160 psi
Deformed bars ..... $u = 0.062 f'_c$ but not to exceed 200 psi

Bearing: f_c

On full area ..... $f_c = 0.25 f'_c$
On one-third area or less‡ ..... $f_c = 0.375 f'_c$

*See Section 407. **See Section 505 and 408.
†Where special anchorage is provided (see Section 503), one and one-half times these values in bond may be used in beams, slabs and one-way footings, but in no case to exceed 200 p.s.i. for plain bars and 250 p.s.i. for deformed bars. The values given for two-way footings include an allowance for special anchorage.
‡The allowable bearing stress on an area greater than one-third but less than the full area shall be interpolated between the values given.

103—Allowable unit stresses in reinforcement

The unit stresses in lb. per sq. in. on reinforcement shall not exceed the following values. Lower unit stresses than those herein specified shall not be used in the calculations for the design of the structure.

(*a*) *Tension in longitudinal and web reinforcement*

Structural grade steel bars ..... $f_s = 20{,}000$ psi
Intermediate grade steel bars, and hard-grade bars (billet steel, rail steel, axle steel and cold drawn steel wire) ..... $f_s = 24{,}000$ psi

(*b*) *Tension on one-way slabs of not more than 12 feet span*

For the main reinforcement, $\frac{3}{8}$ inch or less in diameter, in one-way slabs, 60 per cent of the minimum yield point specified for the particular kind and grade of reinforcement used, but in no case to exceed 30,000 psi.

(*c*) *Compression in vertical column reinforcement*

Forty per cent of the minimum yield point specified for the particular kind and grade of reinforcement used, but in no case to exceed 30,000 psi.

(*d*) *Compression in the metal core of composite and combination columns*

Structural steel sections.....16,000 psi.
Cast iron sections.....10,000 psi.
Steel pipe.....See limitations of Section 706(*b*)

(*e*) *Compression in flexural members*

For compression reinforcement in flexural members see Section 306(*b*)

CHAPTERS 2 TO 8 OF EMERGENCY CODE AND 6 TO 12 OF ACI

The text of chapters 2 to 8 of the Emergency Code are all but identical, section by section, with ACI chapters 6 to 12 respectively with a few *important exceptions:*

All sections bear numbers conforming with the renumbered chapters and most references *within* sections of the ACI Code are changed to conform to the renumbering, while some such references are eliminated from the Emergency Code when they relate to sections in ACI chapters 1 to 5 which are not correspondingly covered in the new chapter 1. Thus Emergency Code

CHAPTER 2—DESIGN—GENERAL CONSIDERATIONS

is numbered Section 200 to 203 (instead of 600 to 603 as in the ACI Code) and the references in Sec. 203 (*c*) line 4, become 102, 103 and 710 (instead of 305, 306, and 1110).

In the following those changes noted in section numbers of the Emergency Code which are directly in consequence of transposing the main provisions of ACI chapters 6 to 12 to Emergency Code chapters 2 to 8 will not be given further note. Changes in *substance* or in *references to section numbers involved in the new provisions of Emergency Code chapter 1,* will be specifically indicated.

CHAPTER 3—FLEXURAL COMPUTATIONS

This follows ACI chapter 7 under the same title except—

Section 305—Requirements for T-beams, (d), eliminates "of Section 505" and "404 (*d*)" from line 3, and "relating," from line 4, of ACI Section 705 (*d*) to read:

(*d*) Provision shall be made for the compressive stress at the support in continuous T-Beam construction, care being taken that the provisions relating to the spacing of bars, and to the placing of concrete shall be fully met.

Sec. 309—Floors with supports on four Sides, (*a*), changes formula numbers of the footnote [2] (ACI 709 (*a*) from (7) to (11) to (6) to (10).

Sec. 309 (*b*) and (*c*) changes formula numbers (2) and (3) to (1) and (2) respectively.

CHAPTER 4—SHEAR AND DIAGONAL TENSION

Sec. 401—Shearing unit stress, (*a*) and thereafter through other chapters formula (11) is ACI (12) and *all subsequent formulas take the next lower number.*

CHAPTER 6—FLAT SLABS—WITH SQUARE OR RECTANGULAR PANELS

Sec. 604—Bending moment coefficients, (*a*), Tables 604 (*a*), (*b*), and (*c*) are ACI Tables 1004(*a*), (*b*), and (*c*).

Sec. 605—Panel strips and critical design sections, (*c*) deletes from the end of ACI 1005(*c*) last line, the reference "as defined in Chapter 1."

Sec. 609—Openings in flat slabs, references in the last line are to Sec. 102 and 103 (instead of 305 and 306 as in ACI).

CHAPTER 7—REINFORCED CONCRETE COLUMNS AND WALLS

Sec. 703—Spirally reinforced columns, (*b*) *Vertical reinforcement*, reads in the first sentence, "The ratio p_g shall not be less than 0.005 nor more than 0.02" instead of "0.01 nor more than 0.08" as in ACI Sec. 1103(*b*).

Sec. 703 (*e*), *Protection of reinforcement*, reads in the final clause, "nor shall it be less than required by fire protection and weathering protection," eliminating the ACI reference (Sec. 1103 (*e*)) to Sec. 507.

Sec. 704—Tied Columns (*a*) *Allowable Load*, the p_g ratio "shall not be less than 0.005 nor more than 0.02" instead of ACI provisions Sec. 1104 (*a*) for 0.01 and 0.04 respectively.

Sec. 705—Composite columns (*c*), *Splices and connections of metal cores*, reference is to Sec. 102 (*a*) (instead of ACI 305 (*a*)).

Sec. 710—Allowable combined axial and bending stress, last line,

$$C = \frac{f_a}{0.35\,f'_c} \text{ (instead of } \frac{f_a}{0.45\,f'_c}\text{)}$$

Sec. 712—Reinforced concrete walls (*a*), last line, the p_g ratio is 0.02 (instead of ACI 0.04).

Sec. 712 (*i*) the footnote references of ACI to expanded metal and welded wire are eliminated.

CHAPTER 8—FOOTINGS

Sec. 802—Loads and reactions, (*a*). ACI references to Sec. 305 and 306 become 102 and 103.

Sec. 805—Shear and bond (*g*) and (*h*) refer to Sec. 102 (instead of 305 in ACI).

Sec. 806—Transfer of stress at base of column (*f*) refers to Table 102 and Section 102 (instead of 305 (*a*) and 305 in ACI).

Sec. 807—Pedestals and footings (*plain concrete*) (*b*) limits tension in concrete to $0.04\,f'_c$ (instead of $.03\,f'_c$ as in ACI).

ACI BUILDING CODE CORRECTIONS

While ACI members and others are making notes in or in reference to their pamphlet copies of ACI Codes issued September 1941, it would be well for them to correct errors in that printing as follows (although having no bearing on the "Emergency Code" in which the corrections are presumably embodied):

p. 7, *204—Portland cement* (*a*): change, in line 2, "C9-38" to C150-41"; and delete the remainder of the paragraph beginning with "or" at the end of line 2.

p. 7, *205—Concrete aggregates* (*a*): change in line 3, "C33-39" to "C33-40."

p. 8, *207—Metal reinforcement* (*d*): line 2, change "Tentative" to "Standard."

p. 12, *305*, Table 305 (*a*): 5 sets of 3 lines each in column 4 "but not to exceed" followed by a psi. value, should appear in column 3 to the left.

p. 23, *700—Notation:* In the line "r_A = Proportion etc." "span A" should be "span A."

p. 28, *709—Floors with supports on four sides*, footnote 2, three mathematical expressions are typographically poor: formula (10a) should read:

$$e_A = \frac{2}{4 - \frac{F_B B}{F_A A}}$$

and formula (10b): $e_B = \dfrac{2}{4 - \dfrac{F_A A}{F_B B}}$

and formula (11) should read: $(1 - r_A)\,\dfrac{wBA}{2}$

p. 32, in formula (14) "sin" should be "*sin*"

p. 33, in formula (15) "sin" and "cos" should be "*sin*" and "*cos*"

p. 33, under *807—Shearing stress in flat slabs:* the first line on p. 34 should read:

"and parallel or concentric with it, shall not exceed the following."

p. 45, under *1005—Panel strips and critical design sections*, paragraph (b) should be indented to conform with (*a*)

p. 45, 1006—*Slab thickness and drop panels* (*c*) $\dfrac{\mathrm{L}}{36}$ should be $\dfrac{L}{36}$

p. 47, *1100—Notation F* should read $F = \dfrac{\textit{Yield point of pipe}}{45{,}000}$ etc.

p. 50, under *1103—Spirally reinforced columns*, (*d*) *Spiral reinforcement*, formula (21) should read $p' = 0.5\left(\dfrac{A_g}{A_c} - 1\right)\dfrac{f'_c}{f'_s}$

p. 54, under 1106, *Combination columns*, (*b*) *Pipe columns*, should read

$$F = \frac{\textit{yield point of pipe}}{45{,}000}$$

p. 60, under *1204—Bending moment* (*g*) formula (30), should read

$$\frac{\textit{Reinforcement in band-width } (B)}{\textit{Total reinforcement in short direction}} = \text{etc.}$$

Discussion of this paper should reach the ACI Secretary in triplicate by Mar. 1, 1943, for publication in the JOURNAL for June, 1943

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Proposed Recommended Stresses for Unreinforced Concrete*

Reported by ACI Committee 322

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GEORGE A. MANEY
DOUGLAS E. PARSONS
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PREFACE

The shortage of steel has turned attention to the use of unreinforced concrete, but little information on this subject is available in such form that it can be readily used by the designer. It was therefore decided that immediate publication of permissible stresses to aid the designer in this emergency was a desirable project, and a committee was formed to make recommendations accordingly.

It was realized at once that unreinforced concrete was not applicable as a substitute for reinforced concrete for all types of construction, for all conditions, and in all locations. On the one hand it was suggested that a list be made of all types of structures that could be constructed of unreinforced concrete. On the other hand it was thought better to prepare a list of structures that could not be built of unreinforced concrete. Committee 322 attempted to make such lists, but shortly discovered that what might be regarded as feasible for one set of conditions would not necessarily appear proper where any one of the variables was changed.

Could a tank to contain liquid be constructed of unreinforced concrete? This has been done, but diameter and height are the limiting conditions. Would a tall chimney be feasible of unreinforced concrete? Masonry has been used repeatedly for this purpose, but where seismic forces are a factor, location as well as size is injected into the problem.

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Could a deep gorge be spanned by unreinforced concrete? Certainly not by beams, but arches have met this test for years and the problem here reduces to a question of type. Could walls be built of unreinforced concrete? They are being built but the spacing of control joints is closer. Detailing is an important factor. Too many variables in different combinations seem to make lists of specific uses impractical.

The effect of volume change is especially important in unreinforced concrete structures. The direct effect of these should be reduced to negligible amounts by providing free movement at closely spaced joints. In structures such as arches and domes where joints providing free movement are not feasible, the secondary effects of volume changes should be estimated as closely as possible and added to those caused by load before applying the allowable stresses which follow.

The allowable stresses are given as functions of both the compressive and flexural strength of test specimens. It is recognized that the ratio of flexural strength to compressive strength is not a constant, but within the usual range of concrete strengths and materials the variation is within limits consistent with other assumptions of design and it is thought that the flexural strength is more representative of actual controlling conditions than is cylinder strength. This test also has the advantage that equipment for making it can be constructed at little cost and is therefore more readily available.

SCOPE

These provisions cover the use of unreinforced concrete and they are to be used in conjunction with "Building Regulations for Reinforced Concrete" (ACI 318-41).

QUALITY OF CONCRETE

As an alternate to the compressive strength as a control for the quality of concrete, the flexural strength may be used. If flexural strength is used as the control, then tests shall be made in accordance with the "Standard Method of Test for Flexural Strength of Concrete" (ASTM C78-39). Curves showing the relation between water content and flexural strength shall be made in the same manner as required for tests involving water content and compressive strength. The requirements for field testing shall be as required by ACI 318-41 for compression tests.

VOLUME CHANGES

Joints, permitting free movement, shall be provided at frequent intervals in straight walls or similar members to reduce to a negligible amount the tensile stresses resulting from volume changes. Such joints shall be spaced not to exceed 16 ft. in members exposed to the weather

and not to exceed 32 ft. in members protected from considerable changes in temperature and moisture content.

Concrete arches and domes shall be designed so as to keep the combined stresses, including those due to volume changes, within the allowable limits.

ALLOWABLE STRESSES

The stresses in concrete shall not exceed the values given in this section where f'_c equals the minimum compressive strength and f'_x equals the minimum flexural strength, both determined at 28 days or at the earlier age at which the concrete may be expected to receive its full load. The stresses given apply to controlled concrete only.

TABLE 1

Description	Allowable Stresses	
	Based on Compressive Strength: f'_c	Based on Flexural Strength: f'_x
Flexure: f_x		
Extreme fiber in tension	.05 f'_c	.33 f'_x
Extreme fiber in tension for arch or dome construction	.08 f'_c	.50 f'_x
Shear: v		
Shallow members $\left(\frac{d}{b}\text{less than } 4\right)$	.02 f'_c	.13 f'_x
Deep members $\left(\frac{d}{b}\text{more than } 4\right)$	.03 f'_c	.20 f'_x
Bearing: f_c		
On full area	.25 f'_c	1.5 f'_x
On 1/3 area or less	.375 f'_c	2.25 f'_x

NOTE: In the design of footings, the lower 2 inches of concrete shall not be included in calculations for stress.

The allowable compressive stress on piers (height not exceeding 3 times least lateral dimension) shall be 0.25 f'_c. Where the height does not exceed 10 times the least lateral dimension, the allowable stress shall be 0.25 f'_c for walls and 0.20 f'_c for columns. For walls or columns of greater unsupported height, the allowable load shall be reduced in accordance with formula 26 of ACI 318-41.

Where reinforcement uniformly spaced at not more than 4 in. centers and weighing not less than 0.3 lb. per sq. ft. of surface area is used, the allowable extreme fiber stress in tension as given in the table may be increased 25 per cent except for arch and dome construction. This steel is to be placed in the direction of stress and not more than 1 in. from the tension surface of flexural members.

Discussion of this report should reach the ACI Secretary in triplicate by Mar. 1, 1943, for publication in the JOURNAL for June, 1943

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Inspection of Concrete Floor Finish Construction*

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SYNOPSIS

The ACI Manual of Concrete Inspection has little discussion of floor finishes and refers to several other sources of information. The references are excellent so far as they go. The present paper has been suggested from several sources, to point out some of the more common defects in finishes and give additional details of procedures which help to prevent the occurrence of such defects.

INTRODUCTION

Among other duties of the building inspector, he is required to see that the floors are built in accordance with the specifications. In concrete floor finish construction there are many things to look out for that are not covered in the usual specifications. One way of bringing these to the attention of the inspector is to point out the more common defects, their causes and the procedure that will help to prevent them.

The most common difficulties are dusting, crazing or checking, cracking, lack of wear resistance and poor bond between courses. Segregation and water gain cause dusting and crazing and when excessive will result in poor wear resistance. Segregation and water gain are usually associated with thick layers of concrete. Actually they can occur in the thinnest layers of concrete or mortar although the amount is usually in proportion to the thickness of layer. This is one of the reasons why a better wearing surface may be expected in two-course than in one-course construction. A five or six-inch slab requires a plastic mix for complete consolidation. At the same time there must be enough fines and mortar on the top for finishing. Working such concrete into place in a single course is almost certain to bring water to the surface. The situation is aggravated by wet mixtures, mixtures that contain too little sand in proportion to coarse aggregate and are therefore, harsh,

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or by aggregates that are poorly graded, particularly if there is a lack of fines in the sand. Floating and troweling the surface do not correct the condition and if done too soon may make it worse.

In spite of these deficiencies, one-course construction is considered good enough for some types of service and is often specified for warehouses, for example, where there will be little trucking and no processing of goods. To get the best job possible the mixture must be of proper proportions, consistency and workability. Water rising to the top surface should be avoided. It is an indication of too wet a mix, insufficient sand or insufficient fine particles in the sand and investigation of the difficulty may be made in that order. With proper proportions a fairly stiff mix can be used, not exceeding 2 or 3 in. slump. If the mix is harsh working at this consistency, a higher sand content should be tried. After screeding to grade, the concrete should be undisturbed until it has stiffened sufficiently so that floating and troweling will not bring water to the surface.

In one-course construction, a dusted-on drying mixture is sometimes specified even though it shouldn't be. The fact that this is permitted does not condone the use of a wet mix in the slab. It is the inspector's job to see that the dusted-on material is uniformly spread in the amount and proportions specified. In this type of work careless placing in the slab will result in wet spots which the contractor attempts to correct by dusting on a large amount of dry materials in these areas and then will use very little or none on the drier areas. The wet spots should be avoided in the first place. The dust coat should be applied soon after the slab is placed and should be thoroughly worked into the slab concrete with floats. If these precautions are not taken, scaling may occur and dusting will be excessive.

SUBGRADES

For all concrete floors built on the ground it is highly desirable that the nature of the subgrade, that is, the type of material, moisture content and compaction of the fill be as uniform as possible. All fill material should be thoroughly compacted by rolling or tamping into place in layers not more than 6 in. thick. Before placing concrete the subgrade should be saturated. This helps to avoid some areas of concrete stiffening more rapidly than others and will produce more uniform conditions for finishing.

TWO-COURSE CONSTRUCTION

For heavy duty service, two-course floors should be provided. The top course may be placed before the base has thoroughly hardened or at any time later. Here again the moisture condition of the base on

which concrete is placed is important. If the top is placed immediately after the base concrete it will absorb water from the base. This would not be uniform and hence some areas of the top surface would stiffen faster than others. It is better to wait 2 or 3 hours until the base has stiffened sufficiently so that men can operate on it without leaving foot depressions. It should then be broomed to remove excess fine material and the top should be placed immediately. The base should be finished to a fairly level surface avoiding depressions and unevenness that would give a topping of varying thickness. Changes in floor level and slopes to drains should be taken up in the base rather than in the topping.

When the topping is to be placed at a later date the base should be broomed to remove fine material and laitance. This should be done when the concrete has hardened sufficiently so that the scum and fine material can be removed without pulling the coarse aggregate out of the concrete. It is important to make certain that, just prior to placing the topping, the base is cleaned of all material that may interfere with bond between the two courses. Plaster, paint, oil, grease, sawdust and just plain dirt may be present and it is up to the inspector to see that these are completely removed. Some contractors specializing in floors sweep up all the debris they can and then use powerful vacuum cleaners to make sure of a perfectly clean surface. The vacuum cleaning is not required by most specifications but is mentioned here to show the extent to which experienced men will go to make certain of a clean surface.

The next requirement is that the base concrete be saturated. No doubt the best practice is to sprinkle the concrete very copiously the day before the top is to be placed, allowing overnight absorption. In a dry, warm atmosphere it may be necessary to sprinkle the slab lightly again just before placing the topping, but in no event should there be pools of water on the slab at the time topping concrete is placed. The slab should be sufficiently wet so that moisture is not drawn rapidly or unevenly from the topping mix. There will be less differential shrinkage between top and bottom courses if the base is saturated. Differences in temperature between the base and finish courses also affect the differential shrinkage. In winter construction it is not good practice to place heated concrete on an extremely cold concrete slab. Subsequent warming of the slab causes expansion and loss of moisture from the top course causes shrinkage, which may cause breaking of bond between the two courses.

A slush coat of cement and water should be broomed into the surface of the wet floor slab. The mixture should be about like paint and should be well brushed out to avoid a thick layer. A necessary precaution is to make sure that the topping mix is placed before the slush coat sets or dries. Thus, this coat should not be placed too far in advance of placing

the topping. If this coating dries before placing the topping it may reduce the bond instead of improving it.

The consistency required for the topping mix will depend on whether floating is to be done by hand or by machine. Sometimes the strike-off boards are long, making necessary a wetter mixture than required for the other operations of finishing. When finishing is to be done by machine, the mix should be so stiff that after spreading and compacting the concrete and striking it off to grade the machines can be operated over it immediately without bringing water to the surface and without sinking into the concrete. Such a mix requires short strike-off boards and usually boards 5 or 6 ft. long and edged with metal are used. These necessitate close spacing of screed strips, but these are usually produced in the concrete itself. Guide blocks are placed at proper elevation at about 10-ft. intervals. After spreading the concrete, a 1-in. board is run over the blocks in a sawing motion, compacting the concrete underneath and forming a screed strip 1 in. wide. Screed strips between these are made in the same way using the screeds at 10-ft. intervals as guides for proper elevation. Excess concrete in the intervening spaces is then scraped off with the short strike-off boards, the surface floated and troweled. When floating is to be done by hand somewhat longer strike-off boards may be used as the mix must be more plastic. Even in this case the mix should be stiff enough to require some exertion in floating to bring enough mortar to the surface for finishing. If water is brought to the surface the floating should be delayed.

Troweling a plastic mix brings water and fine material to the surface, an excess of which may cause crazing and dusting. Troweling should be delayed, therefore, until such time that the operation brings only enough fine material to the top to give a smooth, dense surface. When floating is done by machine the first troweling may follow immediately, but when floating is by hand the troweling usually can be delayed. Final troweling should be done after the concrete has hardened sufficiently so that little if any grout will adhere to the edge of the trowel.

AGGREGATE GRADING AND MIXES

While some fine particles in the aggregate are necessary, too much should be avoided as the fine material tends to build up on the top surface. The wear resistance is thus reduced and there is a greater tendency to craze and dust. Clay, silt and stonedust are especially to be avoided. In general, the sand for the wearing course of two-course construction should contain not more than 5 per cent of particles passing the No. 100 sieve, not more than 15 per cent passing the No. 50 sieve, and from 35 to 55 percent of the particles should be larger than the No. 16 sieve. Coarse aggregate for the topping mix should be graded

between the No. 8 and ⅜-in. sieves, with at least 40 per cent of the particles larger than the No. 4 sieve. Crushed stone consisting of thin, flat particles should be avoided as they give a very harsh mixture difficult to compact and finish.

Cracking of floor finish has often been due to using too rich a mix combined with too high a water content. An advantage of using pea gravel or other coarse aggregate in a topping mix is that leaner mixes with lower water contents are practicable. For hand-placing, a mix of about 1 part cement, 1 part sand and 1½ parts coarse aggregate can be used. For machine floating the coarse aggregate can be increased up to about 2 parts.

CURING

The inspector should see that curing is started as soon as possible without marring the surface. If curing is not started early, surface drying may occur rapidly enough to cause crazing. Atmospheric conditions and methods of finishing will affect the time when curing can be started. In most cases it cannot be started until the morning following the finishing but should not be delayed longer. Floor surfaces outdoors should be protected from sun and wind by placing raised covers over them almost immediately after finishing the surface.

Alternate wetting and drying during curing should be avoided as the expansion and contraction may produce surface crazing. This is especially likely to occur in warm, dry weather and where there are strong air currents, all of which may contribute to rapid drying of the surface between the times of sprinkling. For this reason watertight curing paper or a moisture retaining cover such as sand or sawdust is better than sprinkling which is so likely to be intermittent or sporadic. The sand or sawdust should be placed as soon as possible and should be kept constantly moist until the end of the curing period. Usually it is not necessary to wet the floor before placing the paper if this is put down as soon as possible after finishing the surface. All seams should be sealed and every precaution taken to keep the paper in place.

In cold weather construction, precaution must be taken to avoid drying of concrete near heating elements. Salamanders and similar devices should be raised well off the floor. A thicker moisture retaining cover than usual should be provided on areas near heating apparatus and the necessary precautions taken to be sure that it is kept constantly moist.

Discussion of this paper should reach the ACI Secretary in triplicate by Mar. 1, 1943, for publication in the JOURNAL for June, 1943

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Discussion of a paper by A. J. Boase:

Inspection of Concrete Floor Finish Construction*

By MAURICE COBURN†

Wherever one goes there are rough and worn industrial concrete floors, yet, there is still a surprising lack of attention to this important problem. There is not as yet as complete an adoption of the two-course method as Mr. Boase indicates.

In discussing one-course construction, Mr. Boase says that water coming to the surface should be avoided and he tells us how to do it. The writer has doubts of the possibility, under ordinary conditions, of getting a properly compacted floor slab with maximum strength, without the expectation of some moisture at the top. And there surely must be a decided change from common practice if there is to be no bleeding. Professor Scholer‡ says: "Bleeding, long recognized as an objectionable characteristic of concrete, has its uses, and much of our construction practice depends upon the presence of free water for lubrication of the finishing tools. The arbitrary reduction of the bleeding tendency by the use of finely ground cement or any means except the reduction in the total amount of water used may lead to serious errors in construction practices."

"Dusting-on" has been in disfavor because it has been used to cover up excesses of water and to cut finishing costs improperly. But it can be handled so as to be of real value. It can be used to get more uniform hardness, to produce a drier finish, to finish with an aggregate with better grading and better wearing qualities and at the same time, to reduce finishing cost and the need for effort and expense to reduce bleeding and unevenness in moisture. There has been "dusting-on" that helps in all these respects and that has proven its value in years of extensive use with results that stand comparison with two-course work and for a much lower cost.

*ACI JOURNAL, Nov. 1942; *Proceedings* V. 39, p. 97.
†Sales Engineer, Maximent Corp., Indianapolis, Ind.
‡Consistent Inconsistencies in the Consistency of Concrete, ACI JOURNAL, Apr. 1941; *Proceedings*, V. 37, p. 537.

To get the best results, this calls for a factory prepared material, a selected aggregate, thoroughly dried, carefully sorted and recombined, mixed with the correct amount of cement and sacked, ready for broadcasting on the newly screeded slab in quantity that can get its needed water from underneath. Thus there can be produced a finish that is as lean and dry as can be worked, a result with a minimum of shrinkage, permeability and dusting. For this purpose there is available silica, with rounded particles, that is uniformly hard, tough and acid resistant. It can be used with larger particles and less cement than a crushed material.

When a completely dry mixture is dusted on very slight variations in the amount of absorbed water are readily apparent and can be corrected easily by the addition of dry material to the damp spots. With this treatment a uniform hardness can be had that is unexcelled and the important value of the method is its ability to provide for the surface the uniform hardness that is essential to stand up under heavy trucking. Ordinary sand, more or less damp, cannot accomplish as much and it requires, undesirably, more cement.

Mr. Boase says that, for one-course work where dusting-on is done, wet spots should be avoided. In the instructions concerning two-course work he correctly indicates that variations in the moisture in the slab are to be expected. It is the opinion of the writer that it is not ordinarily economically practicable to avoid such variations, and that in the separate topping there can be variations in hardness due to this cause. A dusting-on finish that is spread uniformly and does not correct variations in moisture by the addition of dry material is not properly applied. Uniformity in moisture is more important than a uniform thickness for the dusted-on material. Enough mixture can be used so that, with ordinary care, the whole area can be satisfactorily covered. Floors are destroyed by impact, rather than by abrasion and any floor that is worn as deep as this finish can be expected to be so rough that it needs repair, no matter what is underneath.

The method has been used, to a considerable extent, on top of a separate topping, to give a harder and more uniform surface, a better aggregate. At various places there has been practical comparison, with and without a separate topping underneath, without any apparent difference in results. It has been suggested that, when a separate topping is used, construction joints can be spaced more widely, with risk of intermediate cracking, because of the greater depth of special material, but if there is trucking across a crack it had better be repaired, with any type of concrete floor.

Between the skillful handling of a factory material and the use of a little neat cement to save time there are various different specifications for "dusting-on." Many of them, if used with the same intelligence and workmanship needed to get results from a separate topping, can improve the quality of the concrete in the finish. Special aggregates, introduced to give increased resistance to abrasion, are of doubtful value where they need more water and cement and are difficult to apply evenly. During the past year hundreds of acres of floors have been laid where, without any increase in cost or only a slight addition a properly applied "dusting-on coat" would have made a very decided improvement.

Mr. Boase says that curing should start as soon as possible. Mr. Swayze has recently told us,* that curing should be delayed until the maximum heat due to hydration has been reached. This very likely is about what we ordinarily get, starting the next morning.

*Early Concrete Volume Changes and Their Control, ACI JOURNAL, Apr., 1942; *Proceedings* V. 38, p 425.

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Influence of Sands, Cements, and Manipulation Upon the Resistance of Concrete to Freezing and Thawing*

By W. C. HANSEN†

SYNOPSIS

This investigation was undertaken to determine the extent to which the resistance of concrete to frost action is influenced by the following variables: (1) Different cements when used with the same sand; (2) Different sands when used with the same cement; (3) Excessive manipulation.

Four sands, four cements, and one coarse aggregate were used in making concretes of four different consistencies. Strengths, absorptions, and resistance to freezing and thawing were determined on all specimens.

The study showed that the influence of both sand and cement on the resistance of concrete to freezing and thawing is determined by their influence on the amount of mixing water required to give the desired consistency and their effect on the bleeding characteristics of the concrete. It was further brought out that manipulation of the surface will not have a marked influence on the resistance if the water-cement ratio in the surface mortar is not increased thereby. A good relationship was shown between the 48-hour absorption of specimens dried 14 days in air and the resistance of the concrete to frost action.

INTRODUCTION

The study to be reported in this paper was undertaken as one phase of an investigation to determine the cause of surface and progressive scaling of concrete pavements. In a paper‡ presented to the Highway Research Board, December, 1940, it was shown, from a study of pavement cores, that the primary cause of scaling was segregation of the concrete during placing. It was also shown that the resistance to freezing and thawing of concrete from different pavements intended to comply with the same specifications varied widely. The investigation reported here was undertaken to determine the extent to which such

*Received by the Institute July 29, 1942.
†Portland Cement Assn., Chicago.
‡W. C. Hansen—Uniformity of Cores an Indication of Pavement Quality, *Proc. Highway Research Board* 20, 568 (1940).

TABLE 1—IDENTIFICATION OF CEMENTS AND AGGREGATES

	Material
Cement Y	Normal portland cement (Lot 15196)
" X	" " " (Lot 15197)
" V	Clinker of 15197 ground with 0.05 per cent Vinsol resin (Lot 15198)
" Z	High early strength portland cement (Lot 15201)
Natural Sand	(Lot 15195) from Tioga, Pa.
Natural Sand	(Lot 15199) from Long Island, New York
Stone Sand	(Lot 15200) Crushed Iron Ore Tailings from Port Henry, N. Y.
Stone Sand	(Lot 15202) Crushed Dolomite from Syracuse, N. Y.
Coarse Aggregate	(Lot 15203) Crushed Dolomite from Beaver Dam, Wisc.

TABLE 2—PHYSICAL DATA FOR AGGREGATES

Material	Lot	Physical Properties						
		Absorp. at 24 Hr. % by Wt.	Water Vapor* Sorption at 18 Hr. % by Wt.	Sp. Gravity		Loss in Wt. Per Cent		Void Content Dry Sand %†
				True	Apparent	10 Cycles Sod. Sulfate	100 Cycles F & T	
Tioga Sand	15195	3.35	1.13	2.720	2.489	14.0	4.0	....
L. I. Sand	15199	0.81	0.78	2.689	2.631	....	2.5	....
Stone Sand	15200	0.52	0.41	3.012	2.965	....	3.6	51.1
Stone Sand	15202	1.13	0.56	2.725	2.644	....	5.1	52.2
Coarse Agg.	15203							
	No. 4¾	1.64	0.64	2.813	2.689	....	....	....
	¾−1½	1.01	....	2.804	2.726	....	0 3	....
	1½+	0.88	....	2.809	2.741	15.0	....	....

*Absorption of sand crushed to minus 100 mesh when exposed to an atmosphere of 95 per cent R. H. at 80 F for 18 hrs.

†Method recommended by National Crushed Stone Association for stone sands. (Information Circular No. 2-28).

Lot	Per Cent Retained on Screen Indicated						
	4	8	16	30	48	100	F. M.
15195	7	33	57	77	89	95	3.59
15199	4	22	39	58	82	95	2.99
15200	4	18	40	64	82	92	3.00
15202	0	11	46	70	85	94	3.06

Lot	Coarse Aggregate Grading Retained on Screen Indicated			
	2½-in.	1½-in.	¾-in.	⅜-in.
15203	0	36	58	98

variations in the resistance of concrete to frost action were influenced by the following variables:

(1) Different cements when used with the same sand.
(2) Different sands when used with the same cement.
(3) Excessive manipulation.

EXPERIMENTAL

Materials

The aggregates used in this study were selected because they have been used extensively in pavement concrete. These materials are identified in Table 1.

The coarse aggregate used in all specimens was a crushed dolomite from Wisconsin, which has a good service record as a coarse aggregate in concrete. Sand 15195 from Tioga, Pa. was composed largely of argillaceous sandstone particles which are fairly readily disintegrated by weathering. Sand 15199, Cow Bay sand from Long Island, N. Y., was composed largely of quartz, feldspar, and granite particles, all of which are very resistant to weathering action. These two sands represent about the extremes in resistance to natural weathering action which are encountered in service. The two stone sands were composed of mineral particles which are very resistant to weathering action. The physical data for all of the aggregates are given in Table 2.

The identifications of the four cements used are given in Table 1, and the physical and chemical data are given in Table 3. Cement V (Lot 15198) was prepared in the laboratory by grinding clinker from the supply used in the manufacture of Cement X (Lot 15197) with .05 per cent Vinsol resin.

Specimens and tests

Two 7x10x30-in. beams were made for each cement and aggregate combination and for each consistency. One of the two beams, identified as specimen A was finished with a wooden float with as little working of the surface as possible. The surface of the other beam, specimen AX, was rodded until the coarse aggregate particles were about ½ in. below the surface and then finished with a wooden float.

The mix used (1-2.78-3.77 by absolute volumes) was the same for all specimens. In the preparation of the specimens, the water-cement ratio was varied as required to give concrete of the desired consistency as measured by the slump in a 12-in. slump cone. The absorptions of the sands, given in Table 2, were measured on samples before the tests were started. The quantity of sand required for the two beams was weighed and put into a measured amount of water 18 hr. before the specimens were made. At the same time a small weighed sample of the

TABLE 3—PHYSICAL AND CHEMICAL DATA FOR CEMENT

Cement	Lot	Chemical Analysis—Per Cent by Weight											Chloroform* soluble
		SiO_2	Al_2O_3	Fe_2O_3	CaO	MgO	SO_3	Ign. Loss	Ins. Res.	Na_2O	K_2O	Free CaO	
Y	15196	21.71	4.96	3.91	65.01	1.22	1.48	0.96	0.17	0.25	0.58	0.75	0.010
X	15197	20.33	6.37	2.83	63.46	2.16	1.96	1.82	0.10	0.19	0.67	1.42	0.007
V	15198	20.88	6.25	3.12	63.80	1.99	1.76	1.20	0.18	0.18	0.57	0.82	0.036
Z	15201	20.39	5.90	2.09	64.19	3.33	2.29	1.15	0.13	0.29	0.38	1.30	0.007

Cement	Lot	Sp. Gr. in Kerosene	Bleeding†		w_i	Surf. Area sq. cm. /gm.	Merriman Sugar Values ml. of .5N NCl	
			Base water Ratio (W/C)b	Rate K x 10^5			Phen.	Clear
Y	15196	3.173	0.91	1360	0.24	1830	25.6	37.8
X	15197	3.128	1.08	1210	0.30	1950	46.1	69.5
V	15198	3.134	....	....	....	1950	38.8	58.5
Z	15201	3.153	1.46	670	0.35	2610	52.5	74.5

Cement	Lot	Expansion during 5 hr. in Autoclave, Cycle Indicated Per Cent (3 hr. at max. temp. 420 F)			Soundness Pat	Time of Set‡				Water for Consistency Per Cent
		1st	2nd	3rd		Initial h.	Initial m.	Final h.	Final m.	
Y	15196	.007	.023	.033	OK	5	10	7	20	25.5
X	15197	.092	.123	.134	OK	3	25	6	30	23.5
V	15198	.101	.136	.153	OK	3	20	6	20	25.5
Z	15201	.119	.165	.183	OK	3	10	5	55	27.0

*Determined to see that the cements 15196, 15197 and 15201 contained no mineral or other oils or fats
†Bulletin 2, P. C. A. Research Laboratory, for definition of terms.
‡Gillmore needle.

sand was dried at 110 C. and reweighed to determine the moisture content of the dry sand. The coarse aggregate was used dry.

When making the specimens, the sand, which had been in water 18 hr., was added to the cement and dry, coarse aggregate in the Lancirick mixer. Sufficient water was added to the mixer to give concrete of the desired slump. The water added to the sand and to the mix plus the water carried by the sand in the original condition was calculated in terms of gallons per sack of cement to give the total water-cement ratio. The total amount of water minus the amount required to saturate the sand is called the net water-cement ratio. The cement and sand combinations studied are given in Table 4, from which it will be seen that Cow Bay sand and Tioga sand were each used with each of

TABLE 4—COMBINATIONS OF CEMENTS AND SANDS STUDIED WITH THE SLUMPS AND WATER-CEMENT RATIOS OF THE CONCRETES

Cement	Lot	Sand 15199				Sand 15195				Sand 15200				Sand 15202			
		Ref. No.	Slump in.	W/C Gal./Sack		Ref. No.	Slump in.	W/C Gal./Sack		Ref. No.	Slump in.	W/C Gal./Sack		Ref. No.	Slump in.	W/C Gal./Sack	
				Total	Net			Total	Net			Total	Net			Total	Net
Y	15196	1	1.3	5.00	4.35	17	0.6	5.91	4.61								
		2	2.9	5.51	4.80	18	3.1	6.31	5.02	33	3.5	5.99	5.33	37	3.5	5.99	5.20
		3	6.0	5.84	5.19	19	6.2	6.72	5.42								
		4	8.5	6.55	5.90	20	8.0	7.60	6.30	34	8.5	7.11	6.46	38	7.5	6.86	6.08
X	15197	5	1.4	5.31	4.66	21	1.2	6.13	4.83								
		6	3.3	5.65	4.99	22	2.8	6.42	5.12								
		7	5.7	5.96	5.31	23	6.0	6.86	5.57								
		8	7.8	6.77	6.12	24	8.0	7.76	6.46								
V	15198	9	1.4	5.08	4.43	25	1.3	5.74	4.44								
		10	2.5	5.42	4.77	26	2.4	5.99	4.69	35	2.8	6.08	5.42	39	3.0	6.08	5.42
		11	7.0	5.82	5.16	27	5.5	6.67	5.37								
		12	7.5	6.64	5.99	28	8.2	7.63	6.33	36	7.3	6.86	6.20	40	8.2	7.08	6.45
Z	15201	13	1.1	5.22	4.56	29	1.4	6.21	4.92								
		14	3.0	5.61	4.96	30	2.5	6.59	5.29								
		15	5.0	6.12	5.47	31	5.7	7.35	6.05								
		16	8.3	7.90	6.35	32	8.0	8.48	7.18								

TABLE 5—STRENGTH AT 28 DAYS AND SPECIFIC GRAVITY OF 7 X 7 X 10-IN. BEAMS

Cement	Ref. No.†	Modulus of Rupture lb. per sq. in.	Comp. Str. lb. per sq. in.	Sp. Grav. Determined from Beam End	Sp. Grav. Calc. from Batch Data *	Ref. No.†	Modulus of Rupture lb. per sq. in. ‡	Comp. Str. lb. per sq. in. ‡	Sp. Grav. Determined from Beam End	Sp. Grav. Calc. from Batch Data
	Sand 15199					Sand 15195				
Y	1A	675	5780	2.489	2.487	17A	780	5020	2.455	2.450
	1AX	695	5870	2.508	"	17AX	675	5110	2.467	"
	2A	570	5580	2.472	2.468	18A	650	5020	2.447	2.433
	2AX	630	4780	2.489	"	18AX	565	4360	2.446	"
	3A	610	5060	2.447	2.450	19A	645	4330	2.434	2.412
	3AX	610	5650	2.478	"	19AX	510	4580	2.424	"
	4A	590	5170	2.486	2.419	20A	500	3590	2.424	2.373
	4AX	485	5660	2.459	"	20AX	450	3250	2.402	"
X	5A	685	6660	2.489	2.473	21A	645	5950	2.430	2.437
	5AX	505	5270	2.466	"	21AX	570	4820	2.444	"
	6A	545	5860	2.473	2.457	22A	550	3890	2.435	2.424
	6AX	575	5540	2.464	"	22AX	555	5250	2.466	"
	7A	605	5650	2.458	2.442	23A	590	4870	2.428	2.404
	7AX	555	5440	2.469	"	23AX	560	3070	2.422	"
	8A	600	5160	2.430	2.405	24A	460	4150	2.417	2.365
	8AX	420	4730	2.455	"	24AX	385	3770	2.372	"
V	9A	510	2730	2.297	2.485	25A	510	3040	2.284	2.456
	9AX	400	2400	2.286	"	25AX	455	3200	2.305	"
	10A	445	2920	2.215	2.469	26A	495	2580	2.251	2.445
	10AX	320	2800	2.208	"	26AX	390	2470	2.245	"
	11A	495	2460	2.203	2.450	27A	330	2380	2.171	2.414
	11AX	345	2150	2.186	"	27AX	250	1840	2.189	"
	12A	435	2550	1.987	2.413	28A	295	3070	2.201	2.371
	12AX	345	3070	2.241	"	28AX	340	2810	2.196	"
Z	13A	705	5000	2.476	2.478	29A	725	5980	2.464	2.433
	13AX	640	5550	2.482	"	29AX	510	4870	2.449	"
	14A	685	5000	2.463	2.452	30A	650	5110	2.430	2.416
	14AX	615	6820	2.469	"	30AX	635	5630	2.420	"
	15A	595	4670	2.438	2.435	31A	605	2800	2.388	2.383
	15AX	595	5740	2.452	"	31AX	445	3980	2.400	"
	16A	560	4160	2.408	2.395	32A	530	3130	2.347	2.335
	16AX	595	4290	2.413	"	32AX	385	3820	2.330	"
	Sand 15200					Sand 15202				
Y	33A	560	4380	2.558	2.535	37A	485	5020	2.463	2.450
	33AX	555	4500	2.559	"	37AX	550	4690	2.482	"
	34A	550	3240	2.533	2.482	38A	410	4420	2.467	2.410
	34AX	505	4220	2.530	"	38AX	485	4600	2.462	"
X	35A	370	2160	2.189	2.549	39A	350	2290	2.302	2.456
	35AX	330	1700	2.165	"	39AX	390	2440	2.307	"
	36A	380	2400	2.207	2.494	40A	365	2840	2.332	2.400
	36AX	245	2070	2.149	"	40AX	290	3240	2.392	"

*Calculated on basis that the coarse aggregate absorbed no water and occupied its dry volume in the hardened concrete.

†See Table 4 for Ref. No. key.

‡Compressive strengths and modulus of ruptures are based on single tests.

the cements in concretes having 1, 3, 6, and 8-in. slumps, whereas the two stone sands were used only with Cements Y and V and in concretes having 3 and 8-in. slumps.

The specimens were cured 14 days in the moist room, followed by 14 days in air of 50 per cent relative humidity at 75 F. The beams were tested dry in flexure at the end of the curing period with center loading on a 28-in. span. One end was then broken in half which gave a piece approximately 7x7x10-in. for use in the freezing and thawing experiments. The other end was tested in compression as a modified cube. The strength data are given in Table 5.

The pieces, broken off for the freezing and thawing tests, were weighed dry, then immersed in water for 48 hr. and weighed again in air and under water to obtain the absorptions and specific gravities. The absorptions during this 48-hr. immersion were calculated and are given in Table 11. Attention is called to the fact that this method of making absorption tests differs materially from the standard procedure usually used. The specimens are not oven-dried at any stage of the test and the 48 hours under water does not produce complete saturation. However, the amount of water absorbed under these conditions has a relation to some property of the concrete which influences durability in service.

The pieces used for the absorption tests were frozen and thawed for 100 cycles in tap water. The rate of disintegration was so slow that the specimens were transferred to a 10 per cent calcium chloride solution and frozen and thawed for another 180 cycles. These were examined and weighed periodically. When a specimen lost 50 per cent of its weight, it was considered to have failed and was discarded. All the weight changes are based upon the dry weight of the specimen before immersion in water. The complete freezing and thawing data are given in Table 11.

During freezing, the metal cans containing the water or chloride solution and the specimens are exposed to the cold air in the freezing chamber. The temperature of the air and the water for a typical freezing cycle are tabulated below.

DISCUSSION OF RESULTS

Table 6 summarizes the results of the absorption and freezing and thawing studies for the concretes made with Cow Bay and Tioga sands with three of the Cements, Y, X, and Z. The data for concretes made with the stone sands and with Cement V ground with Vinsol resin will be presented later. In this table only the "A" specimens (not rodded) are included, and these are arranged, except for specimen 15A, in the order of the absorption. The third column of this table gives, for those

WATER AND AIR TEMPERATURES

Measured by thermocouples in the water between the specimens and in the air of freezer. Specimens placed in freezer at 7:30 a. m.

Time of Reading	Temp. of Water in Containers Degrees F	Temp. of Air in Freezing Chamber Degrees F
7:30 a. m.	63	−15
8:30	36	
9:30	32	
11:30	30	−20
12:30 p. m.	22	
1:30	5	
2:30	0	
3:30	− 5	−25

TABLE 6—SUMMARY OF DATA SHOWING RESISTANCE TO FREEZING AND THAWING IN RELATION TO ABSORPTIONS AND WATER-CEMENT RATIOS

Ref. No.	Abs. % by Wt. 48 hr. in water*	Freezing Cycles at which Loss exceeded 50 per cent	Loss in weight at 280 cycles per cent	W/C gal./sk		Sand	Cement	Slump in.
				Net	Total			
Group A								
32A	1.73	140	....	7.18	8.48	Tioga	Z	8
24A	1.51	180	....	6.46	7.76	"	X	8
31A	1.27	280	....	6.05	7.35	"	Z	6
20A	1.25	180	....	6.30	7.60	"	Y	8
16A	1.12	125	....	6.35	7.00	Cow Bay	Z	8
8A	1.08	200	....	6.12	6.77	"	X	8
23A	1.05	260	....	5.57	6.86	Tioga	X	6
19A	1.04	240	....	5.42	6.72	"	Y	6
15A	0.63	260	....	5.47	6.12	Cow Bay	Z	6
Group B								
30A	0.96	...	3.3	5.29	6.59	Tioga	Z	3
18A	0.94	...	15.3	5.02	6.31	"	Y	3
4A	0.91	...	3.6	5.90	6.55	Cow Bay	Y	8
14A	0.90	...	0.5	4.96	5.61	"	Z	3
17A	0.90	...	3.2	4.61	5.91	Tioga	Y	1
22A	0.90	...	22.4	5.12	6.42	"	X	3
7A	0.81	...	9.4	5.31	5.96	Cow Bay	X	6
21A	0.80	...	1.9	4.83	6.13	Tioga	X	1
29A	0.78	...	2.2	4.92	6.21	"	Z	1
Group C								
6A	0.71	...	+1.5	4.99	5.65	Cow Bay	X	3
2A	0.66	...	+0.9	4.80	5.45	"	Y	3
3A	0.65	...	+1.2	5.19	5.84	"	Y	6
5A	0.61	...	+1.6	4.66	5.31	"	X	1
13A	0.56	...	+0.6	4.56	5.22	"	Z	1
1A	0.55	...	+2.8	4.35	5.00	"	Y	1

*Absorption during 48 hr. immersion in water following 14 days in moist room and 14 days in air of 50 per cent R. H. at 70 F.

specimens which failed, the cycle of freezing and thawing at which the specimens had lost 50 per cent or more by weight. The fourth column gives the weight changes at the end of 280 cycles of freezing and thawing for those specimens which had not failed. Columns 5 and 6 give the net and total water-cement ratios. It will be noted by subtracting the values in column 5 from those in column 6 that Tioga sand in these mixes absorbed an amount of water equivalent to 1.30 gal. per sack of cement and that Cow Bay sand absorbed an amount equivalent to 0.65 gal. per sack of cement.

The specimens in Table 6 are divided into three groups as follows:

A. Specimens which had lost 50 per cent in weight during the 280 cycles of freezing and thawing.

B. Specimens which lost less than 50 per cent in weight during the 280 cycles of freezing and thawing.

C. Specimens which gained weight during the 280 cycles of freezing and thawing.

It will be noted in Table 6 that the three groups of specimens based on the losses in weight also represent varying degrees of absorption. Specimens of high absorption being in group A, intermediate absorption in group B, and low absorption in group C. This division on the basis of absorption is so consistent that the specimens (with a single exception of 15A) were arranged in the order of decreasing absorptions without changing the grouping on the basis of losses in weight. All of the specimens in group A (except specimen 15A) had absorptions greater than 1 per cent. All of the specimens in group B had absorptions between 0.78 and 0.96 per cent and all of the specimens in group C had absorptions less than 0.78 per cent.

It will be noted in Table 6 also, that both the net and total water-cement ratios of the specimens group themselves into high, intermediate, and low values, respectively, in groups A, B, and C. The net water-cement ratios for the group A specimens vary between 7.18 and 5.42; for the group B specimens they vary between 5.31 and 4.83 except for specimen 4A, and for the group C specimens, the water-cement ratios vary between 4.80 and 4.35 except for specimens 6A and 3A. The seemingly abnormal behavior of specimens 4A, 6A, and 3A will be discussed later.

As stated in the introduction, one purpose of this investigation was to determine the influence of sand upon the resistance of concrete to freezing and thawing when used with the same cement.

An examination of the data in Table 6 shows that (making no distinction between cements),

1. When the concrete had an 8-in. slump, all three of the specimens made with Tioga sand and only two of the three specimens made with Cow Bay sand fall in group A.

2. When the concrete had a 6-in. slump, all three specimens made with Tioga sand and only one of the three specimens made with Cow Bay sand fall in group A.

TABLE 7—COMPARISON OF THE NET AND TOTAL WATER-CEMENT RATIOS BY WEIGHT REQUIRED FOR CONCRETES OF THE SAME SLUMPS WHEN REFERRED TO THE WATER-CEMENT RATIOS FOR CEMENT Y AND LONG ISLAND SAND (15199) AS 100

Slump in.	Cement	Net Water-Cement Ratio for		Total Water-Cement Ratio for	
		Cow Bay Sand 15199	Tioga Sand 15195	Cow Bay Sand 15199	Tioga Sand 15195
1	Y	100.0	105.9	100.0	118.2
	X	107.1	108.5	106.2	122.6
	Z	104.9	112.9	104.4	124.2
3	Y	100.0	104.4	100.0	114.5
	X	102.8	106.6	102.5	116.3
	Z	103.3	110.1	101.8	119.4
6	Y	100.0	104.6	100.0	115.0
	X	102.4	107.4	102.0	117.5
	Z	105.4	116.5	104.8	125.9
8	Y	100.0	106.9	100.0	116.0
	X	103.8	109.5	103.3	118.5
	Z	107.6	121.6	106.9	129.4

3. When the concrete had a 3-in. slump, all three specimens made with Tioga sand and only one of the three specimens made with Cow Bay sand fall in group B.

4. When the concrete had a 1-in. slump, the three specimens made with Tioga sand fall in group B and the three specimens made with Cow Bay sand fall in group C. No specimens with Tioga sand fell in group C.

These results clearly show that the characteristics of the sand are important factors in determining the resistance of the concrete to freezing and thawing. The reason for the superiority of Cow Bay sand over Tioga sand when used as fine aggregates in concrete exposed to frost action will be discussed later.

As stated in the introduction also, one object of this investigation was to determine the effect of different cements upon the resistance of concrete to frost action when used with the same sand. An examination of the data in Table 6 shows the distribution tabulated below between the groups A, B, and C for the 8 specimens made with each cement.

	Group A	Group B	Group C
Specimens made with Cement X	3	3	2
" " " " Y	2	3	3
" " " " Z	4	3	1

The obvious conclusion from these freezing and thawing studies is that individual sands and individual cements have an influence upon

the resistance of concrete to frost action. It was shown previously, however, that the resistance of concrete to frost action was definitely related to the absorption of the concrete and that this, in turn, was related to the water-cement ratio used in the preparation of the concrete. Therefore, to find what causes the difference in the resistance of concrete to frost action, when different sands and different cements are used, it is logical to look to the water requirements of the concretes in which these sands and cements are used.

Comparison of the water requirements of the concrete

The concretes used in this investigation were made from a constant mix (by absolute volume), and the water content was allowed to vary as required to give the desired slump. This is a rather common field practice.

Table 7 has been prepared to compare the water requirements of the concretes made with the different sands and the different cements. For this purpose, the water-cement ratio of the concrete made with Cow Bay sand and Cement Y at each of the four slumps is taken as 100, and the water-cement ratios for the Tioga sand and the other cements are expressed as a ratio of this value.

An examination of this table shows that in the case of each of the concretes of the four different slumps, the water requierments, both net and total, for the Tioga sand are very much greater than that for the Cow Bay sand, and the water requirements for Cements X and Z are somewhat greater than that for Cement Y. These data, considered in connection with those in Table 6, show why all the specimens made with Tioga sand fall in groups A and B, and only half of those made with Cow Bay sand fall in these two groups, and further, why all the specimens in group C are those made with Cow Bay sand. These data also show why the three cements used in these tests are not equally represented in the three groups. The difference in resistance to freezing and thawing between these three cements seems clearly to be related to those properties of the cements which influence the water requirements of the concretes.

Effect of bleeding upon the resistance of concrete to frost action

In Table 6, specimen 4A falls in Group B, whereas from its net and total water-cement ratios (5.90 and 6.55, respectively), it appears that it should fall in Group A. Likewise, specimens 6A and 3A fall in Group C, whereas from their net and total water-cement ratios (4.99 and 5.65 respectively for 6A, and 5.19 and 5.84 for 3A), they would be expected to fall in group B.

By reference to Table 3, it is seen that the values for (W/C)b are 0.91, 1.08, and 1.46, respectively, for cements Y, X, and Z. (W/C)b is the

base water-cement ratio, which Powers, in Bulletin 2*, has shown is the theoretical (though not the actual) water-cement ratio at which there would be no bleeding of a cement paste made with the cement in question. The bleeding capacity or the amount of water lost through bleeding at any higher W/C increases as this base water ratio decreases. Therefore, pastes made from Cement Y are seen to have much higher bleeding capacities than those made from either Cements X or Z. Likewise, the bleeding rates, ($Kx10_6$) were 1360, 1210 and 670 respectively for the three cements. Thus Cement Y also has the greatest bleeding rate.

It was observed in the making of the specimens that the concretes made with Cements X and Z were very cohesive and free water separated very slowly, whereas the concretes made with Cement Y were less cohesive and free water separated much faster. This difference in cohesiveness and separation of water increased as the slump increased. Thus, both the laboratory measurements of bleeding rates and capacities, and the observations during the preparation of the specimens, show that the concretes made with Cement Y would be expected when hardened, to have water-cement ratios well below those at which they were placed, whereas concretes made with the other two cements would have, in the hardened condition, water-cement ratios much closer to those at which they were placed. This, decrease in W/C due to greater bleeding reduced the absorption and therefore, explains why specimen 4A falls in Group B instead of Group A; also why specimen 3A falls in Group C instead of Group B.

With respect to the sands, it was observed during the preparation of the specimens that those made with Cow Bay sand tended to separate free water faster and in greater amounts than did those made with Tioga sand. Thus, concretes made with Cow Bay sand which required less water at the start lost more water after placing than did those made with Tioga sand. This probably accounts for a part of the greater resistance to frost action of the specimens made with Cow Bay sand, and may explain why specimen 6A falls in group C instead of in group B.

Tests with stone sands

Table 9, which gives data for the specimens made with stone sand, is similar to Table 6 for the specimens made with the two natural sands. It is seen from Table 9 that one of the four specimens had an absorption of 0.99 per cent and that this specimen had lost over 50 per cent of its weight at 200 cycles of freezing and thawing. The other three specimens had absorptions of less than 0.9 per cent. Two of these specimens showed a slight loss in weight at the end of 280 cycles and one showed

*T. C. Powers—The Bleeding of Portland Cement Paste, Mortar and Concrete, Bulletin 2, Portland Cement Association Research Laboratory (1939).

TABLE 8—COMPARISON OF THE NET AND TOTAL WATER-CEMENT RATIOS AS IN TABLE 7 FOR CEMENTS Y AND V AND INCLUDING THE ARTIFICIAL STONE SANDS

Slump in.	Cement	Net Water-Cement Ratio for				Total Water-Cement Ratio for			
		Cow Bay Sand	Tioga Sand	Stone Sand		Cow Bay Sand	Tioga Sand	Stone Sand	
		15199	15195	15200	15202	15199	15195	15200	15202
1	Y	100.0	105.9	.....	.....	100.0	118.2	.....	.....
3	Y	100.0	104.4	111.0	108.2	100.0	114.5	108.7	108.7
6	Y	100.0	106.9	109.5	103.0	100.0	116.0	108.5	104.9
8	Y	100.0	104.6	.....	.....	100.0	115.0	.....	.....
1	V	101.8	101.8	.....	.....	101.6	114.8	.....	
3	V	99.3	97.6	112.9	112.9	98.4	108.7	110.3	110.3
6	V	99.5	103.5	.....	.....	99.6	114.2	.....	.....
8	V	101.5	107.2	105.1	109.4	101.3	116.5	104.7	108.1

TABLE 9—SUMMARY OF DATA SHOWING RESISTANCE TO FREEZING AND THAWING IN RELATION TO ABSORPTIONS

Specimens made with stone sands 15200 and 15202, and Cement Y.

Ref. No.	Abs. per cent by Wt. 48 hr. in water	Freezing Cycles at which loss exceeded 50 per cent	Loss in weight at 280 cycles per cent	W/C gal./sk		Sand Lot No.	Cement	Slump in.
				Net	Total			
34A	0.99	200	....	6.46	7.11	15200	Y	8
33A	0.82	...	+1.05	5.33	5.99	15200	Y	3
38A	0.84	...	0.73	6.08	6.86	15202	Y	8
37A	0.56	...	0.92	5.20	5.99	15202	Y	3

a slight gain in weight. In general, these weight changes during the freezing and thawing correspond closely to what would be expected on the basis of the absorption data in light of the results obtained with the two natural sands.

Table 8 compares the net and total water-cement ratios of the concretes made with the two stone sands and the two natural sands with Cement Y. The freezing and thawing results, given in Table 9 for the specimens made with the stone sands, show a somewhat greater resistance of these concretes to frost action than would normally be expected

TABLE 10—SUMMARY OF DATA SHOWING RESISTANCE TO FREEZING AND THAWING IN RELATION TO ABSORPTIONS

Specimens made with Cement V and all four sands.

Ref. No.	Abs. per cent by Wt. 48 hr. in water	Freezing Cycles at which loss exceeded 50 per cent	Loss in weight at 280 cycles per cent	W/C gal./sk		Sand Lot No.	Slump in.
				Net	Total		
28A	1.72	....	+3.84	6.33	7.63	Tioga	8
27A	1.46	....	+3.42	5.37	6.67	"	6
25A	1.12	....	+2.52	4.44	5.74	"	1
36A	1.12	....	+1.96	6.20	6.86	15200	8
40A	1.07	....	2.85	6.45	7.08	15202	8
12A	1.05	....	+2.56	5.99	6.64	Cow Bay	8
26A	1.03	....	+2.54	4.69	5.99	Tioga	3
39A	0.86	....	1.45	5.42	6.08	15202	3
35A	0.78	....	+1.68	5.42	6.08	15200	3
11A	0.78	....	+2.18	5.16	5.82	Cow Bay	6
9A	0.65	....	+1.78	4.43	5.08	" "	1
10A	0.63	....	+2.32	4.77	5.42	" "	3

on the basis of the water-cement ratios used in the preparation of the mixes. However, all of these specimens were made with the Cement Y, which, as has been pointed out, had a high bleeding rate and bleeding capacity. It was observed during the making of these specimens with the stone sands that free water separated appreciably. Therefore, the water-cement ratios in the hardened concrete probably were somewhat less than those of the concrete as placed. On this basis these stone sands in combination with Cement Y behaved very much as did Cow Bay sand with this cement, and accordingly appear to be about equal in quality to this natural sand as fine aggregates for use in concrete exposed to frost action. It should be noted that the −50 mesh material was 18 and 15 per cent in the two stone sands, (see Table 2). This fairly high percentage of fine material probably was a factor in producing concretes of good resistance to frost action.

Cement ground with Vinsol resin

Tables 8 and 10 give the data for the specimens made with this cement. It is seen from Table 10 that none of these specimens was disintegrated at 280 cycles of freezing and thawing, although a number of them had high absorptions. Only two of the specimens lost any weight during these freezing and thawing tests. By the use of this cement, it was possible to make specimens with concrete of 8-in. slump using Tioga sand which were much more resistant to frost action than were the specimens made

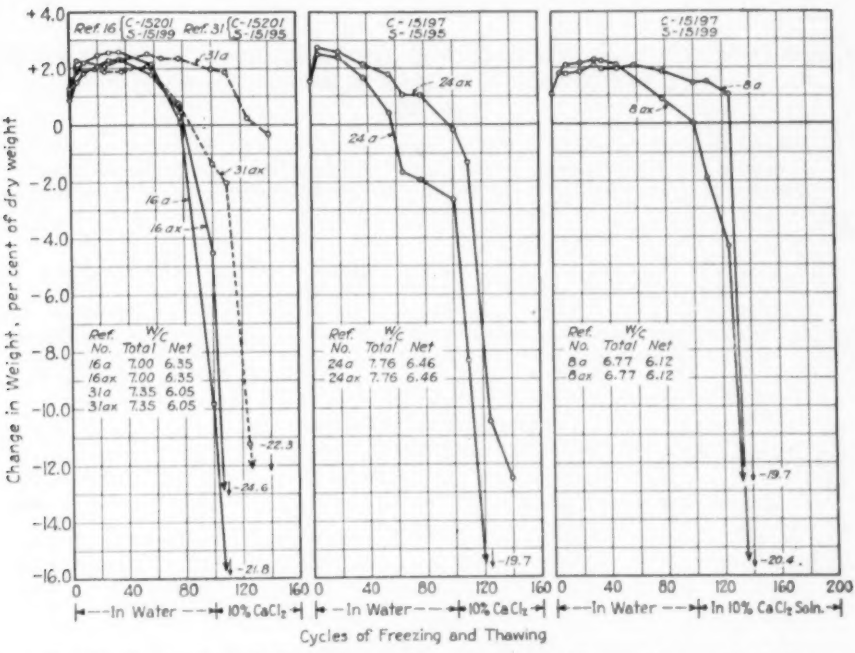

Fig. 1—Comparison of effect of normal manipulation with excessive manipulation on the resistance of concrete to frost action

with concrete of 1-in. slump in which Tioga sand was used with cements which were not ground with Vinsol resin. These results verify the results obtained by others in the laboratory and in the field with cements ground with materials which cause entrainment of air in concrete.

Influence of excessive manipulation upon the resistance of concrete to frost action

The freezing and thawing data obtained with the AX specimens, those which were rodded excessively during placing, are given in Table 11, together with the data for the specimens placed with a minimum of surface manipulation. Plots of the data for four pairs of specimens, 8A and 8AX, 16A and 16AX, 24A and 24AX, and 31A and 31AX are shown in Fig. 1. These data are characteristic of those for the other pairs of specimens. It is seen from this figure that in some cases the two specimens behaved very much alike, in some other cases the AX specimen was slightly less resistant to frost action than was the A specimen, and in still other cases the resistance of the A specimen was slightly less than that of the AX specimen.

The AX specimens were rodded excessively immediately after the molds were filled. This served to force most of the coarse aggregate

TABLE 11—RESULTS OF FREEZING AND THAWING TESTS OF 7 X 7 X 10-IN. SPECIMENS BROKEN FROM 7 X 7 X 30-IN. BEAMS

(Series 286)

Mix by absolute volume: 1-2.78-3.77; specimens cured 14 days moist and 14 days in air; all weights are based on dry weight after 14 days air curing; Frozen and thawed for 100 cycles in tap water and continued in a 10 per cent calcium chloride solution.

Spec. No.	W/C* Gal. per Sack		Absorption 2 days Immersion in water Per Cent	Gain in Weight—Per Cent—After Cycles of Freezing and Thawing					
				In Tap Water			In 10% $CaCl_2$ Solution		
	Total	Net		5	10	100	140	200	280†
Cement Y—Sand 15199									
1A	5.00	4.35	0.55	1.06	1.16	1.10	1.32	1.18	1.11
1AX			0.59	1.05	1.14	1.14	1.30	1.07	0.92
2A	5.51	4.80	0.66	1.05	1.12	1.12	1.45	1.28	1.00
2AX			0.63	1.15	1.25	1.26	1.42	0.98	0.94
3A	5.84	5.19	0.65	1.07	1.12	1.19	1.55	1.12	1.18
3AX			0.83	1.32	1.39	1.37	1.53	1.62	1.48
4A	6.55	5.90	0.91	1.49	1.62	1.49	1.88	0.80	3.63
4AX			0.61	1.28	1.52	2.26	1.93	1.59	0.47
Cement X—Sand 15199									
5A	5.31	4.66	0.61	1.09	1.28	1.21	1.56	1.74	1.58
5AX			0.65	1.18	1.32	1.48	1.55	1.78	1.73
6A	5.65	4.99	0.71	1.34	1.43	1.29	1.43	1.52	1.46
6AX			0.80	1.41	1.50	1.67	1.92	1.68	1.62
7A	5.96	5.31	0.81	1.36	1.46	1.04	0.97	2.01	9.37
7AX			0.91	1.49	1.65	1.75	1.83	1.71	0.93
8A	6.77	6.12	1.08	1.78	1.78	1.44	−2.04	−53.00	(200)
8AX			1.10	1.83	2.08	0.00	−1.97	....	(160)
Cement V—Sand 15199									
9A	5.08	4.43	0.65	....	1.11	1.66	1.62	1.78	1.78
9AX			0.72	....	1.14	1.48	1.81	1.51	1.49
10A	5.42	4.77	0.63	....	1.12	2.01	2.19	2.19	2.32
10AX			0.89	....	1.44	2.25	2.08	2.44	2.25
11A	5.82	5.16	0.78	....	1.47	1.87	1.79	2.10	2.18
11AX			0.83	....	1.56	2.42	2.54	2.58	2.82
12A	6.64	5.99	....	....	2.01	2.47	2.61	2.56	2.56
12AX			1.05	....	2.00	2.16	2.30	2.40	2.42
Cement Z—Sand 15199									
13A	5.22	4.56	0.56	0.98	1.11	1.35	1.57	1.22	0.58
13AX			0.36	0.80	0.91	1.02	1.14	1.09	1.11
14A	5.61	4.96	0.40	1.06	1.23	1.34	1.39	0.88	−0.46
14AX			0.50	1.12	1.19	1.47	1.56	0.48	−0.42
15A	6.12	5.47	0.63	1.13	1.55	1.80	−2.57	−28.6	(260)
15AX			0.62	1.21	1.40	1.63	−0.03	−23.6	(260)
16A	7.00	6.35	1.12	1.89	2.19	−9.82	....	....	(125)
16AX			0.87	1.50	7.81	−4.51	....	....	(125)

*W/C as used in mix uncorrected for any change due to bleeding.
†Numbers in parentheses indicate cycles at which the loss in weight exceeded 50 per cent.

TABLE 11—RESULTS OF FREEZING AND THAWING TESTS OF 7 X 7 X 10-IN. SPECIMENS BROKEN FROM 7 X 7 X 30-IN. BEAMS (Cont'd.)

(Series 286)

Mix by absolute volume: 1-2.78-3.77; specimens cured 14 days moist and 14 days in air; all weights are based on dry weight after 14 days air curing; Frozen and thawed for 100 cycles in tap water and continued in a 10 per cent calcium chloride solution.

Spec. No.	W/C* Gal. per Sack		Absorption 2 days Immersion in water	Gain in Weight—Per Cent—After Cycles of Freezing and Thawing					
				In Tap Water			In 10% $CaCl_2$ Solution		
	Total	Net	Per Cent	5	10	100	140	200	280†
Cement Y—Sand 15195									
17A	5.91	4.61	0.90	1.59	....	0.83	0.35	−1.83	−3.15
17AX			0.87	1.76	....	1.21	0.40	−3.33	−9.12
18A	6.31	5.02	0.94	....	....	0.66	−0.81	−5.40	−15.3
18AX			1.04	1.32	....	0.00	−0.32	−3.92	−26.9
19A	6.72	5.42	1.04	1.95	....	1.44	−0.07	....	(240)
19AX			0.95	1.84	....	1.81	0.42	−10.49	280
20A	7.60	6.30	1.25	2.06	....	1.50	−1.58	....	(180)
20AX			1.32	2.19	....	0.00	−5.22	−20.57	280
Cement X—Sand 15195									
21A	6.13	4.83	0.80	1.79	....	0.89	0.85	−1.21	−1.92
21AX			0.89	1.87	....	1.40	0.58	−18.6	−22.4
22A	6.42	5.12	0.90	1.83	....	2.07	2.01	0.36	−10.76
22AX			0.92	1.97	....	1.44	−6.05	(160)	(160)
23A	6.86	5.57	1.05	2.09	....	−1.97	−6.43	−32.9	(260)
23AX			1.07	2.02	....	−0.36	−7.35	....	(160)
24A	7.76	6.46	1.51	2.49	....	−2.64	−21.8	....	(160)
24AX			1.55	2.71	....	−0.18	−12.5	....	(160)
Cement V—Sand 15195									
25A	5.74	4.44	1.12	1.80	....	2.44	2.52	2.62	2.52
25AX			0.92	1.59	....	2.53	2.42	2.56	2.40
26A	5.99	4.69	1.03	1.81	....	2.29	2.50	2.54	2.54
26AX			1.01	1.91	....	2.50	2.50	2.64	2.64
27A	6.67	5.37	1.46	2.65	....	3.23	3.42	3.42	3.42
27AX			1.38	2.48	....	3.24	3.10	3.25	3.27
28A	7.63	6.33	1.72	....	2.95	3.39	3.56	3.72	3.84
28AX			1.65	....	2.89	3.22	2.96	3.24	3.34
Cement Z—Sand 15195									
29A	6.21	4.92	0.78	1.58	....	1.36	1.48	−1.12	−2.19
29AX			0.86	1.73	....	1.11	1.41	....	−7.13
30A	6.59	5.29	0.96	1.83	....	1.84	1.84	−0.14	−3.31
30AX			0.89	1.81	....	0.90	0.50	−0.14	−5.20
31A	7.35	6.05	1.27	2.24	....	1.98	−0.31	−16.7	280
31AX			1.09	1.96	....	−1.33	−22.3	....	(180)
32A	8.48	7.18	1.73	2.82	....	−12.59	−76.2	....	(140)
32AX			1.76	2.49	....	−17.03	−56.4	....	(125)

*W/C as used in mix uncorrected for any change due to bleeding.
†Numbers in parentheses indicate cycles at which the loss in weight exceeded 50 per cent.

TABLE 11—RESULTS OF FREEZING AND THAWING TESTS OF 7 X 7 X 10-IN. SPECIMENS BROKEN FROM 7 X 7 X 30-IN. BEAMS (Cont'd.)
(Series 286)

Mix by absolute volume: 1-2.78-3.77; specimens cured 14 days moist and 14 days air curing; all weights are based on dry weight after 14 days air curing; Frozen and thawed for 100 cycles in tap water and continued in a 10 per cent calcium chloride solution.

Spec. No.	W/C* Gal. per Sack		Absorption 2 days Immersion in water Per Cent	Gain in Weight—Per Cent—After Cycles of Freezing and Thawing					
				In Tap Water			In 10% $CaCl_2$ Solution		
	Total	Net		5	10	100	140	200	280†
Cement Y—Sand 15200									
33A	5.99	5.33	0.82	....	1.32	1.32	1.39	1.24	1.05
33AX			1.22	....	1.81	2.00	1.94	1.76	1.29
34A	7.11	6.46	0.99	....	1.66	2.08	2.20	−1.40	(200)
34AX			0.89	....	1.72	1.59	1.63	−2.84	(260)
Cement V—Sand 15200									
35A	6.08	5.42	0.78	1.92	1.92	2.42	2.20	1.82	1.68
35AX			0.91	2.21	2.34	2.86	2.94	2.78	2.25
36A	6.86	6.20	1.12	2.17	2.17	2.67	2.84	2.54	1.96
36AX			0.96	2.04	2.04	2.62	3.10	2.91	2.15
Cement Y—Sand 15202									
37A	5.99	5.20	0.56	....	1.15	1.30	1.32	1.15	−0.92
37AX			0.65	....	1.26	1.44	1.56	−1.44	1.44
38A	6.86	6.08	0.84	....	1.55	1.50	1.30	−0.98	−0.73
38AX			0.72	....	1.45	1.29	1.24	0.70	−1.90
Cement V—Sand 15202									
39A	6.08	5.42	0.86	1.56	1.56	1.81	1.60	0.96	−1.45
39AX			0.79	1.65	1.95	2.15	2.00	1.70	−0.86
40A	7.08	6.45	1.07	2.00	2.05	2.32	2.01	0.73	−2.85
40AX			1.19	2.11	2.16	2.20	1.68	1.00	−4.17

*W/C as used in mix uncorrected for any change due to bleeding.
†Numbers in parentheses indicate cycles at which the loss in weight exceeded 50 per cent.

out of the top ½ in. of the surface, producing surfaces on these specimens which consisted largely of mortar. This type of excessive manipulation appears from the results given in Table 11 to have had very little, if any, influence upon the resistance of the specimens to frost action. It should be pointed out, however, that it is not safe to conclude from these studies that excessive manipulation of the surface under all circumstances will not influence the resistance of the concrete to frost action. The only effect of the excessive rodding probably was to force the coarse aggregate out of the top surface mortar without materially changing either the water-cement ratio or the sand-cement ratio.

CONCLUSIONS

In a procedure in which concrete is prepared from a fixed mix by volume with water content variable to give a desired consistency, the following conclusions appear justified from this study:

1. Different cements will produce concrete having different degrees of resistance to frost action if there are appreciable differences in the amounts of water required to give the specified slump, or if the cements produce concretes having different bleeding characteristics.

2. Different sands will produce concretes having different degrees of resistance to frost action if there are appreciable differences in the amounts of water required to give the specified slump, or if the sands produce concretes having different bleeding characteristics.

3. Manipulation of the surface of the concrete slab as carried out in these tests (if the water-cement ratio of the mortar is not increased by the manipulation) will not have so marked an influence upon the resistance of concrete to frost action as will variations in the water requirements and bleeding characteristics of the cements and sands.

4. The absorption of a concrete specimen, as measured in these studies, (not by the usual procedure) is a good criterion of the resistance of the concrete to frost action.

Discussion of this paper should reach the A. C. I. Secretary in triplicate by Mar. 1, 1943, for publication in the JOURNAL for June, 1943

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Discussion of a paper by W. C. Hansen:

Influence of Sands, Cements, and Manipulation Upon the Resistance of Concrete to Freezing and Thawing*

By A. D. CONROW, M. SPINDEL and AUTHOR

By A. D. CONROW†

During the past 20 years, first with State highway testing laboratories of both Kansas and Missouri, and for the last 14 years as research engineer for a portland cement manufacturer, I have observed many tests for durability of concrete to freezing and thawing and have followed rather closely the tests made under the supervision of C. H. Scholer at the Kansas State College, and F. V. Reagel at the Missouri State Highway testing laboratory at Jefferson City.

Many freezing and thawing tests of concrete have been made in the laboratory, under my supervision. I have observed conditions which may cast serious doubts on the interpretation of the resistance of concrete to freezing and thawing, as given by Mr. Hansen. He has used the loss of weight of the concrete specimens as the measure of relative resistance to freezing and thawing of various concretes. Our experience has demonstrated that loss of weight is of little value as a measure of the destructive effect of this treatment.

We have had many specimens having compression strengths as high as 5000 psi at start of tests, so reduced that they could be crumbled with the hands, and in this state there was practically no loss in weight. On the contrary, some specimens show a steady gain in weight, due to greater absorption as the concrete becomes more spongy from the freezing and thawing action.

Practically all of our tests have been made on 3 x 3 x 11-in. beams or prisms with stainless steel gauge points inserted in the ends giving an effective gauge length of 10 in.

I have before me the data for a test during which the alternate freezing and thawing cycles were continued until the concrete became so weak that it could be broken and crumbled with the hands. The expansion

*ACI JOURNAL, Nov. 1942; *Proceedings* V. 39, p. 105.
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amounted to .997 per cent, yet there was only a superficial scaling, the weight loss of which was fully compensated by gain in water absorption. The strength of this concrete at 28 days and one year ages, obtained on cylinder specimens made from the same mix, was found to be: 28 days, 4631 psi; 1 year, 5939 psi. The freezing and thawing tests were started when concrete was 90 days old. The weights of specimens saturated at start and completion of test, were: Start of test, 3864 grams; finish of test, 3865 grams.

We have had many specimens which lost considerable material from their surfaces, but we could not consider this loss a measure of relative injury for in some cases the material lost by a specimen reduced to so spongy a state that it could be crumbled with the hands, was no greater than the loss of others in which the interior concrete appeared to be as sound as at the start of the tests, with only the surface affected.

These experiences lead me to question whether Mr. Hansen may use the weight loss as a measure of resistance to freezing and thawing, as reported in his paper.

Acceptance of Mr. Hansen's conclusion No. 1 would lead to the selection of a cement with high bleeding capacity as best suited for use in concrete to be subjected to severe frost action.

Cements treated with Vinsol resin and other agents producing similar results when used in concrete, yield concretes with very low bleeding capacities, yet these concretes show very high resistance to freezing and thawing. This same conclusion would cause the selection of the cement requiring the least water per bag of cement in the concrete to obtain concrete having a high resistance to freezing and thawing. This conclusion is not borne out when considering concrete of the character obtained when using a cement producing concrete of a character similar to that obtained by the use of Vinsol resin cement. We have observed extremely high resistance to freezing and thawing in concrete with quite high water cement ratio when the concrete was of a character such as produced by the use of "treated cements." Thus there are other factors to be considered than low water cement ratio paste in the concrete.

The resistance of portland cement concrete to alternate freezing and thawing seems to be importantly connected with the incorporation of minute air voids, distributed with fair uniformity throughout the mass of hardened concrete. There is no doubt that the lower water-cement ratio pastes will show greater resistance to freezing and thawing, but there are other factors apparently of equal or greater importance. An illustration of this was observed during the course of our freezing and thawing testing. During some of our early testing, our laboratory was not equipped with a fog curing room for test specimens. We did, however, have a controlled temperature water bath. To obtain what we

considered ideal curing conditions, our specimens were cured immersed in water. They were removed from the molds and immediately placed in the water curing tank. After a three month curing period, the concrete was subjected to freezing and thawing cycles in saturated condition in air. We were amazed how quickly the concrete was destroyed. After only 30 cycles concrete, having 5000 psi compression strength at start of test, had an expansion of 1.0 per cent and was in a state that it could be crumbled with the hands.

This concrete contained a cement-water paste equivalent to 6.5 gal. of water per bag of cement. We later tested other concrete made from the same mix and using aggregates from the same lot. After a curing period in a controlled temperature fog room, this concrete withstood more than 100 cycles without appreciable damage and an expansion less than occurs from a temperature change within the range of normal outdoor exposure in the central United States.

We can account for this difference only on the theory that in the case of the first concrete, all voids were completely filled with water, whereas in the second concrete, at least some of the voids became air filled and no reasonable period of immersion later would refill these voids with water.

Reagel of Missouri found this same effect with a porous aggregate. This aggregate once dried, could not be resaturated except by employment of extraordinary methods. This is reason to believe hardened concrete will perform in a manner similar to that of the aggregate.

This extemperaneous discussion without reference to specific data, is given because of a belief that a too narrow interpretation might be placed on the results of the data Mr. Hansen presented, and on his conclusions.

By M. SPINDEL*

As a former Member of the International Association for Testing Materials I was also a Member of various committees on the weathering quality of building stones and of concrete, and had to deal with the complex problem of porosity and water absorption from various angles. In the last year I made some absorption tests on specimens which had been dried in the laboratory air only, without any heating, and then immersed in water for periods of 5 minutes, 1 hour, 24 hours, 7 days, and partly after previous boiling also up to 4 weeks. These tests were repeated at least four times on the same specimens. Interesting and instructive absorption curves will be published later. I discuss the matter here only as far as it has been dealt with in the paper.

It was very interesting but not surprising that the absorption measured in a way which differs from the standard procedure has proved a good criterion of the resistance of concrete to frost action. Any heating or boiling of the specimens might have had an additional influence on the

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resistance of concrete to frost and aggressives. It is understandable, too, that the amount of water absorbed under the described conditions had a good relation, not only to the water-cement ratio of the mix, but also to the reduced, and, therefore, improved water-cement ratio caused by bleeding, which proved to be of great advantage. In principle the "Conclusions" of the paper are in good accordance with my experience in the laboratory and in the field, and I add some remarks only:

1. The conclusions apply to a concrete of good workability. As the slump test is often not the most suitable measure of workability of concrete I suggest referring not only to the "specified slump" but also to mixes of "nearly the same workability."

2. As to the "bleeding characteristics" there is no doubt about the fact that the less water a workable concrete contains before setting the less is its water absorption and the higher is its resistance to frost etc. Still, the bleeding is of great advantage only if it does not increase the laitance, the starting point of disintegration by weather and frost apart of many other disadvantages due to lack of adhesion, softness, high permeability, shrinkage, etc. I had the opportunity of testing a special addition to concrete which reduced the water-cement ratio of the mix for the same slump and or workability up to 20 per cent with a further decrease of the water-cement ratio by bleeding without giving any laitance. Instead, the surface of the concrete was much harder than without the addition and it adhered completely to the concrete placed above it, even after an interruption of many days.

The highly improved density and strength, reduced water absorption and consequently the much higher resistance to weather and frost which have been obtained by the forementioned addition are in very good accordance with the "Conclusions" of the paper. I referred to the effect of this kind of addition on cement and concrete in my paper on "Special Cements" submitted to the Congress On Large Dams, Washington, 1936.

3. As to the cement V ground with Vinsol resin we shall have to consider which and how much of the other valuable qualities of concrete, such as density, strength, resistance to wear etc., have to be sacrificed in order to obtain a higher resistance to frost as stated in the paper. From table 5 of the paper we may see that since Vinsol resin had been ground to cement X, the compression strength of the concrete was reduced as much as 60 per cent less than that without resin. It may take some time to solve this complex problem.

As to the special methods of testing the absorption as described in the paper:

1. Probably it was due to present circumstances that there were insufficient specimens for an average of at least three results for each test.

2. It is known that some types of natural stones might change their

physical constitution if heated for a long time even at a moderate temperature far below the boiling point of water. Such stones had to be dried at a temperature not exceeding 85 F (30 C) if repeated absorption tests were made, otherwise at a temperature of 122 F (50 C). For concrete it proved an advantage, too, not to heat the samples either in hot air or by boiling if they are to be used afterwards for testing their resistance to frost. Still it would be very useful to determine their probable percentage of moisture before immersing in water on separate samples to be cured in the same way as the specimens used for freezing and thawing.

3. It is known that the resistance to frost action of natural stones or concrete depends not only on the amount of water absorbed after a certain period or after having been saturated more or less completely, but also on the speed of absorption from which the so called "saturation coefficient" is determined. I shall try to point out why the absorption after a period of 48 hours as described in the paper might be considered as a suitable measure of the "saturation coefficient" and thus also as a good measure of the resistance to frost.

4. As a rule the sizes of the samples for absorption tests are small so that at the end of a period of immersion the water could be absorbed more or less completely by the whole sample. The samples were extraordinarily large and, therefore, the absorptions both as to the amount and to the speed differed very much from the usual results obtained according to standard procedure. With regard to the size of the samples the amount of absorption might be considered as an average for absorption periods from 48 hours to a few hours or minutes only and that is why it might be considered somehow as a measure for the "saturation coefficient" which is known to be a very good measure of the resistance to frost. With smaller samples the limits will differ for the different degrees of absorption.

5. Some colleagues are still in doubt whether the artificial weathering cycle does in fact reproduce the effects of natural weathering on a comparative scale. In my opinion it is not necessary to continue the freezing and thawing until the loss of weight is 50 per cent, but it would be an advantage to have some other criterions of the resistance to frost before any loss of weight can be stated. It is known that it could be done by measuring the extension, the decrease of Young's modulus etc.

AUTHOR'S CLOSURE

Mr. Conrow questions the use of loss in weight as a measure of the resistance of concrete to freezing and thawing and points out that there are many factors which have an influence upon this resistance.

In any study of concrete, it is almost impossible to limit the investigation to a study of one variable. In this investigation the purpose was to study the relative influence of either cements or sands when used in concretes of fixed proportions by weight and constant consistency as

measured by the slump. In order to meet the consistency requirement, it was necessary to vary the water-cement ratio. This procedure was followed because a number of specifications call for constant mix proportions and consistency and permit the water-cement ratio to vary. Thus in this investigation either cement or sand was the independent variable and water-cement ratio was the dependent variable.

In investigations of this nature, besides the selection of the variables of mix proportions etc., there are always the questions of the type of specimen and the method of measuring the degree of disintegration. Small specimens permit the use of a greater number of specimens and more refined measurements than are possible with large specimens. On the other hand there is always the question as to whether or not the behavior of a small specimen represents the behavior to be expected of a concrete in service. Careful consideration was given to the matters of type of specimen and methods of measuring disintegration before the selections were made for this study.

The decision to use loss in weight as a measure of the resistance to freezing and thawing in this particular study seemed justified in the light of previous experience in this laboratory and the experience of others. For example, C. H. Scholer (Kansas State College Bulletin 28, 24 (1931)) in a discussion of the loss in weight method states, "For certain types of disintegration produced by cherty limestone and gravel it will be of little value, as for these materials the first indication of failure is a breaking down of the entire mass. When the failure is due to the breaking up of the cement paste, the method would seem to have real possibilities." The coarse aggregate used in these specimens was known, from its service record, to be a durable material, hence it was expected that the failure of the specimens would be a disintegration of the mortar, and this was so.

An investigation giving comparable results of weight losses, linear expansion and flexural strength is the cooperative study reported by H. S. Mattimore (Proc. Highway Research Board 16, 135 (1936)). In this study, which was made for the purpose of determining the influence of cements and other factors upon the resistance of mortars to freezing and thawing, the University of Wisconsin studied 1:2 and 1:4½ prisms during freezing and thawing and measured weight loss, linear expansion, and flexural strength. In Fig. 13 of the report, both linear expansion and weight loss are plotted against cycles of freezing and thawing. An examination of this figure shows a great similarity between the two plots for any one cement, and a study of the data shows that the cements would be rated in almost the same order, with respect to the resistance of the specimens to freezing and thawing, whether relative flexural strength, linear expansion, or loss in weight was used as the criterion of resistance. In Fig. 12 of the report, plots are given comparing the results obtained by the

University of Wisconsin and the Portland Cement Association laboratory for the loss in weight of 1:2 mortars. This figure shows a very good agreement between the results from the two laboratories, which indicates that such results are fairly reproductible.

C. A. Hughes (Proc. Am. Soc. Testing Mat. 33 (II) 511 (1933)) shows in Fig. 5 a fair relationship between linear expansion and loss in weight for mortar prisms.

F. V. Reagel (Proc. Highway Research Board 20, 587 (1940)), in discussing loss in weight and linear expansions, states "It may be found from further investigation that either or both of these measurements may serve as a check on strength measurements or even as an alternate method of determining deterioration."

Mr. Conrow relates experiences with specimens which gained in weight and became spongy and with others which could be crumbled with the hands. A detail of the testing in this investigation, which should have been described in the author's paper, was that each specimen before being weighed was scraped with a knife to remove all loose and spongy material. None of the specimens cracked or showed any signs of disintegration other than sloughing of the surface and loss in weight. The specimens which had not disintegrated during freezing and thawing were stored outdoors for observation of their behavior under natural weathering.

On receipt of Mr. Conrow's discussion these specimens, which had been exposed to two winters of natural weathering, in addition to the 280 cycles of freezing and thawing in the laboratory, were examined. All of the specimens from Group B of Table 6 were found to have undergone some additional disintegration during these two years of natural exposure. Group B specimens, it will be recalled, were those which had shown losses in weight from 0 to 22 per cent in the 280 cycles of freezing and thawing. While no precise measure of disintegration was attempted, specimens in the group which had shown the greatest losses at the time of test had shown the greater change in condition at the time of this examination.

The specimens in Group C (those which had shown a gain in weight during 280 cycles of freezing and thawing) showed little if any disintegration during the two year's weathering. The three showing the least change were brought to the laboratory for test. By way of comparison, three of the specimens from Table 10 were also brought in for test. These specimens were those made with a treated cement, *V*. These three specimens and the three from Group C, Table 6, which all had about the same external appearance, were tested for compressive strength. For results of the tests of these six specimens, together with the 28-day strength of the corresponding concretes, as given in the paper, see Table A.

It will be seen by the increase in strength (average 45 per cent) for specimens 1A, 2A, and 3A that these concretes were not damaged at all

TABLE A

Spec. No.	Net w/c gal./sk.	Compressive Strength, psi	
		28 Days	After 280 Cycles of F. and T. in Lab. and 2 Winters Outdoor Exposure
1A	4.35	5780	8755
2A	4.80	5580	8400
3A	5.19	5060	8140
9A	4.43	2730	4890
11A	5.16	2920	3045
12A	5.99	2460	4180

by either the freezing and thawing in the laboratory or the two years' subsequent outdoor exposure. This, coupled with the continued disintegration, obvious from surface appearance, for the specimens which had shown small losses in weight in the 280 cycles of freezing and thawing (Group B) clearly supports the interpretation of the loss in weight data as given in the paper.

The foregoing statement is further confirmed by the preformance of specimens 9A, 11A, and 12A. These specimens, made with a treated cement, which had low compressive strength at 28 days because of the entrained air, showed no loss in weight in the 280 cycles of freezing and thawing. The two years' further exposure brought no change in surface appearance, but a satisfactory increase in strength is indicated by the tests after the outdoor exposure.

After these six specimens were broken, the pieces were examined for evidence of disintegration during the freezing and thawing in the calcium chloride. No such evidence could be detected nor were there any signs that they have suffered deterioration of any kind. Certainly, if there were any specimens in this series of the kind described by Mr. Conrow as showing no loss in weight and yet having no integrity in quality, they would have been disclosed by the treatment they have been given. Hence, in the light of the performance of these specimens, the decision to use loss in weight as a measure of disintegration due to freezing and thawing in this investigation where deterioration was due to failure of the mortar seems to have been sound.

Mr. Spindel has presented some very interesting and instructive comments especially on the subject of absorption. These add materially to the knowledge of the subject of absorption which is of paramount importance in any consideration of the suitability of materials for structures to be subjected to frost action. It is gratifying to note his general agreement with the conclusions in the paper.

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Job Problems and Practice

"JPP" is a means toward realizing more fully than in a limited number of longer JOURNAL contributions, the mutuality of ACI Membership effort to do a better concrete job. In JPP many Members may participate in few pages. So, if you have a question, ask it. If an answer is of likely general interest, it will be briefed here (with authorship credit unless the contributor prefers not). But don't wait for a question. If you know of a concrete problem solved—in field, laboratory, factory, or office—or if you are moved to constructive comment or criticism, obey the impulse; jot it down for JPP. Remember these pages are for informal and sometimes tentative fragments—not the "copper-riveted" conclusiveness of formal treatises. "Answers" to questions do not carry ACI authority; they represent the efforts of Members to add their bits to the sum of ACI Member knowledge of concrete "know-how."

Steel—Too Much or Too Little (39-111)

The following paragraph was lifted from a recent letter from an ACI member:

"Structural design, just now, requires the utmost in ingenuity. I have had to replace steel gratings in catwalks with precast concrete slabs and stair treads. I have had to make extensive use of the cantilevering possibilities in reinforced concrete, and latterly, have been trying to find ways of getting along without reinforcing steel. Having always been a very moderate user of reinforcing and a lover of ingenuity, you might think I am in clover. Quite the reverse is true. My superiors, who a year ago were reinforcing-mad, and scared to think of a design that wasn't in a text-book, are now no-reinforcing-mad, and scaring me stiff with their boldness. This is a very dangerous condition, and one that the technical societies should be thinking about. It is just as bad engineering to use too little as too much, and just as bad to use too much as too little. This is one reason why I chose this time to join the Institute.

Let us economize all we can in using building materials of all kinds. In so doing, we shall, inevitably, have some failures. Our business should be to achieve the maximum of economy with the minimum of failure. Zeal for economy will be worse than useless unless well placed."

Prestressed Steel in Reinforced Concrete* (39-112)

By F. E. RICHART†

Q—For what use is pre-stressing practical in the light of present knowledge and economic conditions?

A—I might mention some of the uses of pre-stressed steel in reinforced concrete construction that have come to my attention. Probably one of the first examples is the Hewitt system. In a paper before the American Concrete Institute in 1923‡, Mr. Hewitt described his system of using pre-stressed steel in building circular water tanks. The principle is simple, First a plain concrete cylindrical tank was constructed (the one described was 20 ft. in internal diameter and 32 ft. high). After the side walls were finished, horizontal hoops made of 11/16-in. bar were assembled. Each hoop carried three turnbuckles which were tightened by hand to produce an initial tension of about 15,000 psi. in the bars, and a corresponding compression in the concrete shell. Later an outer layer of concrete about 3 in. thick was cast on the outside of the tank, and it was noted that a layer of Gunite might also have been used for this purpose. It is evident that this method would be effective in preventing cracking of the walls due to water pressure in the tank. The same system has been used recently on oil storage tanks, I am told, by the Navy Department.

There are other methods that have been used commercially. A very elaborate system was devised by Mr. Freyssinet in France in 1936.** He used pre-stressed steel in reinforced concrete pipe and in one special case in large hollow cylindrical piles or caissons which were cast in place. He employed several very unusual features in the fabrication of these piles. The reinforcing steel was placed in the form of longitudinal bars and of circular hoops, and the inner form or mold was so arranged that when steam pressure was applied this mold could be pushed outward against the concrete. This pressure was transmitted through the concrete to the reinforcing steel and served to put initial tension in the hooping. At the same time, the longitudinal thrust of the steam pressure acting on the ends of the inner form produced a tensile stress in the longitudinal bars. Add to this the fact that both the heat from the steam used and vibration were employed to produce a high early strength in the concrete, and you have an example of a very unusual application of pre-stressed steel.

I might mention briefly another application of pre-stressing in the manufacture of reinforced concrete pipe. Such a system is used by the

*From ACI "Quiz Session," ACI 1942 Convention.

†Research Professor of Engineering Materials, University of Illinois, Urbana.

‡"A New Method of Constructing Reinforced Concrete Water Tanks," by W. S. Hewitt, *Proc.* ACI, Vol. XIX, p. 41-52, 1923.

**"A Revolution in the Technique of the Utilization of Concrete," by E. Freyssinet, *The Structural Engineer*, London, Vol. XIV. No. 5, p. 242-262, May 1936.

Lewistown Pipe Co. of Fort Wayne, Indiana, and tests have been made by Prof. R. B. Crepps at Purdue University.*

I believe in all of these methods the important thing is that tensile cracks are inhibited by virtue of the compressive stresses that are set up in the concrete. I believe that such construction will meet most of the objections to the use of higher stresses in reinforcing steel, since such objections have usually been to the resulting large cracks and deflections of flexural members which occur under the usual type of design and which would be greatly minimized by a system in which pre-stressed steel is utilized.

Frozen Concrete† (39-113)

Q—In the event that concrete freezes during construction, (a) what is the procedure to save the work? (b) Does a single freezing damage concrete from the standpoint of durability?

By WILFRID SCHNARR‡

A—The answer to this is very short; reverse the process, thaw the frozen concrete. Concrete that has been frozen should be enclosed with tarpaulins or any other suitable enclosure and live steam applied. After the concrete is completely thawed, standard methods of curing should follow.

(Chairman Crum assigned the second part of the question to Mr. Young.)

By R. B. YOUNG‡

I can be even more brief; it does. (to which Mr. Crum asked, "Why?") I do not know the exact why, i.e., the physics and chemistry of the reaction; but I do know a single freezing almost always affects the strength and durability of a concrete frozen while still plastic. Compressive strength is usually reduced by 10 to 50 per cent. Concrete that has been actually frozen while plastic tells its story by its internal appearance; it has through it little turkey tracks or whatever you want to call them, made by the formation of ice crystals and wherever you find these exposed under severe conditions, you will also find disintegration.

By HARRY C. SHIELDS**

If memory serves me correctly, an article appeared in *Engineering News Record* several years ago describing cold weather concreting on a huge construction project in the Russian Ukraine, where they faced the problem of concreting in sub-zero temperatures. Aggregates were heated

*"An Investigation of Stresses in Pre-Stressed Reinforced Concrete Pipes," by R. B. Crepps, Bulletin No. 46, Purdue University.
†From 1942 ACI Convention "Quiz Session."
‡Hydro-Electric Power Com. of Ontario, Toronto.
**Marquette Cement Mfg. Co., Chicago, Ill.

only for the purpose of thawing frozen lumps; mixing water was not heated. Concrete was placed in shallow lifts and was deliberately permitted to freeze rapidly. Success of the operation hinged on freezing the concrete as placed and keeping it frozen until the weather moderated, after which it was thawed. The article stated that this procedure had no damaging effect on the quality or normal physical properties of the concrete and that on thawing it developed normal strength and other desired qualities.

The author of this article stressed the point that success of the method hinged largely on freezing the concrete *before* initial set of the cement took place.

I do not wish to convey the impression that I am recommending nor suggesting this as a winter concreting procedure. I am merely passing it along as an item of interesting reference and record.

By MR. YOUNG

I have read the article that Mr. Shields refers to. It is very intriguing, and I have made several attempts to obtain like results in the laboratory but have failed. I have never been able to freeze concrete in a way that did not get internal segregation of the mixing water into ice crystals. As long as that condition exists, I do not see how we can expect to have concrete as durable as it would otherwise have been, unless something is done to reconsolidate the concrete and eliminate the pores left by these ice crystals. Further, in our tests we have never been able to get anything like 95 per cent of the original strength out of a test cylinder that was first quick-frozen, then thawed out and cured normally.

By H. C. WATTS*

I wanted to cite one experience with concrete, that I did not see put in, as bearing on the question of durability. A concrete slab on the ground froze just after it was freshly poured at a temperature of eight above zero. I thought it was ruined. When I got back to that city 14 years later I asked how long the slab had lasted, and they said "There it is." This concrete was in good condition. It has been my observation with the coarser ground cements that they will seldom stand freezing, but with those finer ground there is better than a fifty-fifty chance of concrete "coming back" and giving good durability.

By N. H. WITHEY†

The Research Laboratory of the Portland Cement Association conducted a series of tests eight years ago. bearing on this question. The

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†Portland Cement Association, Chicago.

results were published in the ACI JOURNAL*. The entire series involved tests of some 6000 concrete cylinders.

When specimens were placed in the freezer at periods of 6 and 24 hours after they were made, and then at the age of 3 to 28 days were brought out of the freezer and placed in water at 72 F, the compressive strength at 3 months was 85 to 100 per cent of the strength of similar cylinders moist-cured the entire time. This has nothing to do with durability, but refers only to the effect on compressive strength of freezing *after* a short, preliminary warm storage period.

By E. A. HAGY†

In the year 1930, in the construction of a hotel in Cincinnati, we had a condition of about three weeks of close to zero weather and we poured footings of three to five yards. We did not make any attempt whatever to protect the concrete, we just poured it and let it freeze and after three weeks it thawed out and the concrete seemed just as plastic as when we poured it. Then we commenced to take care of it, after it thawed once We had made cylinders at the time it was poured and protected them all. We made cylinders from this frozen concrete after it became plastic and cured them, and after a six months period, there was not over a five per-cent difference in the compressive strength of the two concretes. I am citing that as an experience that we had. I believe that no damage was caused by the concrete freezing in the plastic state.

Effect of Vinsol Resin on Bond Between Concrete and Steel (39-114)

Q—What is the effect of Vinsol resin on the bond between concrete and steel?

By CHARLES E. WUERPEL‡

A—1. Tests performed in this laboratory on the bond developed between concrete containing various cements and deformed bars of reinforcing steel, suggest the following conclusions:

a. The use of up to 0.035 per cent of Vinsol resin in portland cement in properly designed concrete mixtures will not result in a significant reduction in the bond between the hardened concrete and reinforcing steel.

b. The use of Vinsol resin in amounts greater than 0.035 per cent appears to reduce the bond in some rough proportion to the amount of Vinsol resin present in the cement. In none of the tests performed by this office was the bond resistance of Vinsol resin cement less than 90 per cent of that of a similar untreated portland cement.

*Jan.-Feb. 1934, *Proceedings* V. 30, p. 159: "Temperature Effects on Compressive Strength of Concrete," by A. G. Timms and N. H. Withey and discussion Nov.-Dec., 1934, V. 31, p. 8.

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‡Senior Engineer, War Dept., Central Concrete Laboratory, Mount Vernon, N. Y.

The tests were performed under the following conditions:

Specimens: 6 x 6 x 10-in. concrete prisms; bars: ⅝-in. round deformed bars (diamond deformations); imbedment: 10 in.; aggregate: glacial sand and gravel (1 in. max.); age at test, 21 days; curing: 14 days in water at 72 F., 7 days in air at 72 F.; rounds: 2 specimens of each type on each of 3 days; determination of bond: pull-out performed on 100,000 lb. Reihle screw type machine. Slip measured on free end of bar by dial calibrated to 1/10,000 in. Slip at 3, 5, and 10 x 10^{-4}; total specimens: 112; positions: ½ vertical; ½ horizontal.

The actual test data indicated a relative bond resistance of Vinsol resin treated cement of from 90 to 113 per cent of similar portland cements.

It is believed important to state that all other data collected by this office indicates that a maximum advantage is to be obtained from the use of Vinsol resin in amounts between 0.025 and 0.035 per cent.

Insulating Value of Cinder-Concrete Masonry Walls (39-115)

Q—Can you tell me the insulating values of hollow cinder-concrete masonry units? Is the modern way of using these blocks for insulation as good as using the rock wool between studs?

A—Relative insulating values of typical 8-in. walls of cinder-concrete block and typical wood frame construction walls are shown in computed heat transfer coefficients for several such walls. These are from Tables 3 and 5 of Chapter 4 (Heat Transmission Coefficients and Tables) of the 1942 Guide of the American Society of Heating & Ventilating Engineers. The coefficients are expressed in BTU per hour per square foot per degree F difference in temperature between the air on the two sides, and are based on a wind velocity of 15 miles per hour.

8-in. cinder block wall

plain wall, no interior finish	0.42
plaster (½ in.) on wall	0.39
plaster on wood lath, furred	0.27
plaster (½ in.) on ½ in. rigid insulation, furred	0.20

Frame construction—wood siding on clapboards, 1 in. wood sheathing

plaster on wood lath on studding	0.25
plaster (½ in.) on ½ in. rigid insulation on studding	0.19
plaster on lath on studding—flexible insulation (1 in.) between studding (two air spaces)	0.12
plaster on lath on studding—rock wool fill (3⅝ in.) between studding	0.07

The tables referred to list coefficients for these same walls with other types of interior finish, as well as for other types of walls.

Shrinkage of Concrete Exposed to the Weather (39-116)

By J. C. PEARSON and R. F. ADAMS*

The accompanying diagram shows the length changes of six pairs of pebble-concrete bars, each 2 x 2 x 10 in. exposed to the weather for three years at the Lehigh Portland Cement Co.'s laboratory near Allentown. The six different cements used were those distributed to a number of laboratories in 1939 for a series of cooperative tests sponsored by Committee C-1 of American Society for Testing Materials. Three were designated as "standard" cements, and three as high early strength cements, but one of the first group was what is now known as Type II cement, the other two being Type I†.

The pebble concretes were all proportioned in such manner as to have the same water-cement ratio (6 gal. per sack) and the same consistency (about 5-in. slump). Because of the different water requirements of the cements, the cement factor varied from about 6.2 sacks per yard for the Type II cement to 6.9 sacks per yard for the finest of the Type III cements.

*Lehigh Portland Cement Co., Allentown, Pa.

†The chemical analyses of these cements are given in the 1940 Report of Committee C-1, Proc. of ASTM, Vol. 40, p. 221, 1940.

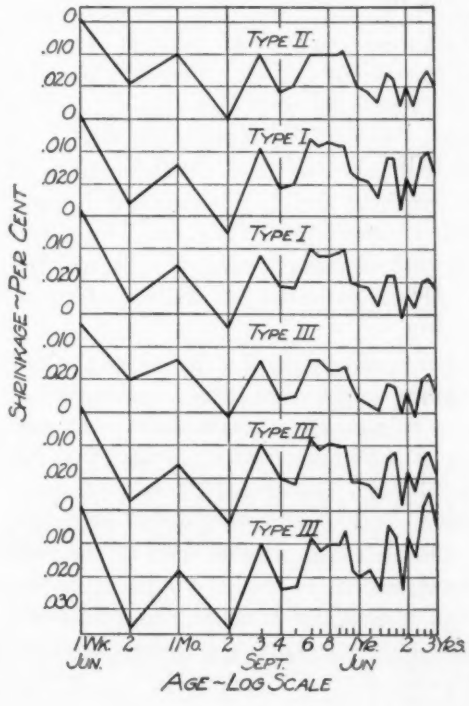

Fig. 1—Length changes in six pairs of pebble concrete bars

The test bars were cured 7 days in the damp room and were then stored out-of-doors on a flat roof (June, 1939), fully exposed to the weather. The "zero" readings of the bars were taken when they were put outside; subsequent length measurements were taken after 2 weeks, 4 weeks, and monthly thereafter.

In spite of the considerable differences in composition and fineness of the cements used, it is seen that all the graphs are remarkably similar and with one exception might well be graphs for different bars of identical concrete. The one exception is the last curve on the diagram which shows slightly wider fluctuations during the first few months, and an upward trend during the second and third years. This gradual expansion may be an indication of comparatively low weather resistance for this particular cement, which is not indicated, however, by the recorded oxide analysis. The fluctuations of the curves are, of course, due to the variable weather conditions prevailing just before the successive measurements were made.

It is interesting to contemplate that these bars, which have a relatively high surface-volume ratio, should exhibit such uniformity in shrinkage and such a low maximum shrinkage. If these bars had been exposed continuously during this time to a relative humidity of 50 per cent, it seems almost a certainty that the Type III bars, made with cements of high fineness and in richer mixtures, would have shown considerably higher shrinkage than the bars containing Type I and Type II cements. This leads to the thought that the shrinkage of different concretes exposed to successive periods of alternating dry and damp weather may be much more uniform than the usual laboratory tests would indicate, the reason being that the concrete becomes damp quickly in wet weather, dries slowly in fair weather, and is not exposed (except rarely) to long drying periods of low humidity. Thus also the richer mixes, having potentially higher drying shrinkage, may show about the same shrinkage under variable weather exposure as leaner mixtures, because they do not dry out as readily.

One other point may be worth mentioning, and that is the method of proportioning mixtures of this sort for comparative tests. On the equal water-cement ratio basis it has been pointed out that in these relatively strong mixtures the cement factors of the bars made with the Type III cements are higher than those made with the Type I and Type II cements, whereas the more natural practice in actual construction would be to use less, or at least, not any more, of the premium cement than of the normal. Therefore it would seem more logical to make comparative exposure tests on the basis of constant cement factor rather than constant water-cement ratio. This procedure is now generally followed in durability investigations, but not universally so.

Variation in Test Strength of Concrete and Mortar Specimens According to Moisture Content at Time of Test (39-117)

By F. L. FITZPATRICK*

In the ACI Manual of Concrete Inspection, p. 90, under the subheading, "Strength Tests", it is stated that a tension or flexure specimen will give perhaps 25 per cent *lower* test result, when tested dry, than when tested damp.

We do not know upon what special experience the above statement is based, but our own rather extensive experience is all to the contrary.

In the course of tension testing of many thousands of briquettes of sand-cement mortar, our experience is that briquettes tested in a perfectly dry state will register from 25 per cent up to sometimes 50 per cent higher results than precisely similar specimens (as to compression and curing) tested in a saturated state.

Similarly, in the practical work of concrete product making, more particularly pipes, we have always found that concrete in a wet state gives a lower modulus of rupture, is more easily cut and worked, and generally is lower in strength than the same concrete when perfectly dry.

In many of our factories, one of our control tests on concrete is the test for modulus of rupture of small beams. Here again, we find that the dry specimens normally give higher strength.

Of course, there may be cases where, due to drying shrinkage, some cracking has occurred which would lower the apparent test strength. For sound specimens, however (at least where the specimens are comparatively small and thin), the dry specimens, according to our experience, give much higher strength results in tension and flexure, as well as in compression and shear values.

Typical of our average run of test results is the following now before me from a current series:

Mix

300 parts by weight standard sand
100 parts by weight portland cement
40 parts by weight water

Tensile Strength—Average of 12 specimens

Stored 1 day damp air, 27 days water.....340 psi.
Stored 1 day damp air, 24 days water, and 3 days air.....447 psi.

Lea and Desch ("Chemistry of Portland Cement Concrete", page 233), dealing with tensile strength, state that the ratio

$$\frac{\text{"wet condition strength"}}{\text{"dry condition strength"}}$$

varies from 0.6 to 0.9 "and even more widely."

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Gonnerman and Shuman, "Compression, Flexure and Tension Tests on Plain Concrete", (Proc. A.S.T.M. 1928, Vol. 28, Part 2, pp. 527-573) report results that at first sight may appear to contradict the foregoing. However, the conclusions to be drawn from that paper are:

1. Specimens stored continuously in damp air give higher strengths than those stored a short time in damp air and then the rest of the period in dry air.

2. Concretes stored continuously in water for a lengthy period give higher strengths than those of similar age stored for a shorter period in water and a considerable later period in dry air.

This is rational as hydration proceeds in the damp stored specimens but is largely suspended in the case of samples stored dry for a long time.

In reinforced concrete units the composite structure may yield better results when tested wet, than when tested dry, but this is due to two factors, viz:

1. The concrete expands when wet, is restrained by the reinforcing steel, and so is thrown into a state of compression or "pre-stressed".

2. Bond value improves somewhat in wet concrete—probably due to swelling, and therefore closer contact between concrete and steel.

It is not the higher strength of the concrete, but rather in spite of lower strength, that a reinforced structure in this condition gives high strength.

I think the ACI Manual statement referred to is incorrect in general application to mortar and concrete.

Causes and Control of Bleeding (38-93)

By JOHN C. SPRAGUE*

Referring to a previous discussion of bleeding (JPP 38-93, Apr. 1942 JOURNAL): In the middle 30's the writer initiated tests to determine what quantities of different sub-sieve fines were required in a concrete mix to limit the bleeding to that obtained in a mix in which the aggregate contained a given amount of natural sand passing a 100 mesh screen. A number of commercial admixtures were included in the test program with the result that, using natural river sand of minus hundred size as the datum with a value of unity, the amount required to produce equal bleeding with other fine material ranged from 0.11 to 1.08 (of unity). In other words, eleven parts of one fine material would have the same effect upon bleeding of concrete as would 108 parts of another material, to take the extremes brought out by the materials used in these tests.

For *predicting* the quantity of different types and kinds of sub-sieve fines to use to produce the same bleeding as was obtained with the natural river sub-sieve sand, the following equation was used:

*Dravo Corp., Pittsburgh.

$X = \frac{ab}{cd}$ where:

X per cent by weight required of the admixture
a specific gravity of natural fines
b normal consistency of natural fines
c specific gravity of the admixture
d normal consistency of the admixture

The use of this equation saved time and material which would otherwise be consumed in starting from "scratch" with trial batches; as it was, one or two trial batches sufficed. That the calculated and actual percentages required in each instance were not radically different is evident upon comparison (see "Evaluating Fines in Concrete on a Bleeding Test Basis", *Proc.* ACI JOURNAL, Sept.-Oct., 1936, V. 33, p. 29).

Mixed Brands of Cement on the Same Job (39-118)

Q—What are the effects of mixing different brands of cement in a single unit of a structure, on the strength, workability, and finish of the concrete in, let us say floors poured monolithic and finished without topping, with a steel trowel?

Recently we placed 5,000 cu. yd. of concrete in about 100 floors and the cement company from which we bought had operating difficulties and shipped a large portion of our order from two other mills in widely separated sections of the country without advising us or the owner. All shipments were in the first company's own bags, therefore we do not know just how, nor where the three different brands were mixed in the job. The specifications prohibited the using of more than one brand on the work, and the owner is still unaware of the mixture.

Many cylinders broke under the requirement of 2500 psi for 28 days and 1600 psi for 7 days. There were soft spots in the floors that had to be removed and replaced. We had trouble satisfying the owner with the finish, due to roughness and off color. More than half the floor area had to be ground to some extent with terrazzo machines.

We are charging this extra cost back to the cement people as we believe they are responsible; we also contend that they did not handle the change properly in the fact that they did not advise us, nor ask the consent of the owner.

The three brands of cement were shipped intermittently throughout the job from start to finish. The cement company has not denied liability and, apparently, the matter will be settled peaceably, and without court action. Have you information on similar experiences?

A—The foregoing inquiry from a contractor was referred to several members of the Institute. The replies supplied the basis of the following:

The matter of liability of the cement company is a legal problem on which four Institute correspondents appear to have no opinion. From one of these we quote:

"It is a well-known fact that different brands of cement all meeting the same specification exhibit remarkably different characteristics as regards color, strength, workability, tendency towards bleeding, time at which concrete surfaces may be troweled and the ease with which such surfaces may be finished. This is particularly true if the cement is purchased under the old ASTM specification or if the cement is purchased as Type 1 under the new specifications (ASTM Designation C150-41 adopted as standard on September 2, 1941) since these specifications have no chemical limitations except as to magnesium oxide and sulphur trioxide.

"I don't know the outcome of a case similar to that described, but I do know that where more than one brand of cement has been employed on large jobs, blending of the several brands of cement has been done to eliminate the difficulties."

From another correspondent:

"We *prefer* big projects to blend cements from several mills. There are data to support the belief that a blend is better than the individual cements; variations in each are smoothed out with the result that the final cement is more uniform and is an aid to our concrete control. If the individual brands were used separately and intermittently, variations greater than that occurring in one brand alone would be noticed.

"By blending, the uniformity in strength, workability, finish of concrete, color, and other characteristics are improved. The objection to the use of several brands of cement in floors where color might be an item of consideration, is understandable. Explanations for the soft spots and low strengths are, of course, not possible without much more information."

From another source this comment: "Various brands of cement can be blended and have been blended on jobs repeatedly—Boulder Dam, an outstanding example. Blending brands of good cement can cause no ill effects. The blending of several good brands with a brand which happens to be faulty may cause trouble."

From other sources come these comments: "The use of different brands on the same structures gives rise to doubts about color uniformity. The properties of the concrete otherwise do not differ in any major way if the separate cements used are of the same type; that is, the type referred to in the classification now used in ASTM specification C150-41."

It is pointed out that in general types 1 and 3 are likely to be light in color. Types 2, 4 and 5 dark in color.

One correspondent pointed out that it is conceivable that in handling cement from two or three different sources on a job of ordinary size that the time might come when half a floor panel could be placed with light cement and the rest with dark cement. This could be annoying to many building owners, particularly while the concrete is fresh and the color difference is most pronounced. These differences tend to disappear in time. In examining structures several years old where it has been important to know the areas in which different cements were used it was found difficult to determine.

In big work it is commonly required that when the source of the cement changes the break be made at some logical division point in the structure to avoid haphazard coloring.

It is also pointed out that the inquirer may be unduly concerned about the results; that perhaps the best thing to do is to take the matter up with the company furnishing the cement.

Light Weight Aggregate to Save Steel (39-119)*

By FRANK E. RICHART†

Q—Would significant steel savings result from the use of light-weight aggregate instead of heavy aggregate?

A—I believe the question is whether one could make a significant saving in reinforcing steel through the use of lightweight aggregates instead of ordinary heavy aggregates. Certainly there may be considerable saving in dead load, since lightweight concrete of excellent quality for structural use can be made which will weigh only 95 to 100 lb. per cu. ft., about two-thirds the weight of gravel or stone concrete. This would reduce the dead load of the structural frame by one-third and this would produce a decided saving, particularly in columns and footings, and to some extent in flexural members.

Concrete made with lightweight aggregate has a very low modulus of elasticity, and this lowers the neutral axis in a beam or slab, producing a reduction (perhaps 4 to 5 per cent) in the moment arm of the resisting couple, or a corresponding increase in steel area required for a given bending moment. For example, if the dead load on a flexural member were three-fourths of the live load, the bending moment might be decreased by 14 per cent by using lightweight instead of ordinary concrete, but the resulting saving in steel would probably be only about 10 per cent. This is on the assumption that the two kinds of concrete would be used in the same depth of slab or beam, and that the two concretes would be of equal strength. Another possibility of saving in total cost might lie in the use of thinner slabs with the lightweight concrete, but

*Question and answers are from the Quiz Session, 38th Annual Convention ACI, February 1942.
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this would result in an increase in the steel requirements. Normally, slabs made with lightweight concrete will call for a large percentage of steel for balanced reinforcement, but if the object is to save steel, there is no reason why a smaller steel percentage should not be used.

It appears then that the greatest saving in steel due to the use of lightweight concrete would be in the columns and footings of a building, but that some saving is also possible in the steel requirements for beams and slabs.

By H. F. THOMSON*

As I am not a designer, I would not be able to discuss the details Professor Richart mentioned. However, a 12-story apartment hotel was built in St. Louis about eight years ago in which light-weight aggregate was used for fireproofing the steel frame.

The statement was made at the time that the saving in cost of steel and foundation piers resulting from the reduction in weight of concrete, offset the increased cost of the cement and aggregate by a substantial amount. The owners were pleased, especially as the reduction in column section materially improved the utility of the lower floors, so far as the actual cost was concerned.

The question has been raised as to the relative cost of light-weight concrete itself as compared to regular concrete. As we happen to have a haydite production adjacent to our industrial district, our company handles a considerable amount of haydite for structural purposes. As between gravel and haydite, the cost of concrete delivered is approximately $4.00 per cu. yd. more for haydite than for gravel on the basis of the same cement content. That differential interpreted in terms of the cost of gravel concrete, represents an increase of about 50 to 60 per cent. When we add the cost of additional cement necessary to duplicate the strength of gravel concrete, that percentage will be increased to approximately seventy-five per cent. This increase refers to the cost of the concrete alone, delivered to the site. The substitution of light-weight concrete does not materially affect the cost of form work, placing and other items of that sort. Thus, although the relative cost of material delivered appears to show a rather important increase in cost, when that increase is expressed as a percentage of the gross cost of the concrete in place, it is not so very large. It is my understanding that, on some types of design, the light-weight producers are able to show a decided economy by the over-all use of the light-weight material for a building of considerable height, such as the one I have mentioned.

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A Method of Mix Control for Light Weight Aggregates (39-120)

By H. G. COLLINS*

When expanded or porous aggregate is used in concrete the technician attempting to control the mixes has his troubles. The usual methods involve determination of absorption and free moisture, in order to correct the batch weights and control the water-cement ratio. Both determinations are extremely difficult with aggregates having a rough, open surface, and the accuracy of the results may be questioned. Further these determinations are time consuming, and the inevitable time-lag discounts their value for control purposes. The method described herein, while not precise, offers a practical means of control that is relatively simple and fast, and, in most of the cases tried, gives satisfactory reproducibility.

A basic assumption is made that a given volume of the aggregate in uniformly compacted condition will yield a uniform amount of concrete. The proposed method is for the purpose of determining the weight of aggregate, as it is in the bin or stock pile, required to make a cubic foot of uniformly compacted aggregate. This weight will, of course, vary with changes in moisture content and density. For lack of a better term, this volume of uniformly compacted material has been called a "constant cubic foot" to distinguish it from "damp, loose," "dry, rodded," etc., or, for brevity, a "C.C.F." To arrive at the number of C.C.F. for a batch is no problem to a technician, but depends on the job and the requirements to be met. With that quantity established, batch weights may be corrected as necessary.

This method does not predetermine the quantity of water to be used. However, consistency is sensitive to water, and judgment of consistency by an experienced man affords a reasonably reliable control of water if the batch quantity of aggregate can be kept uniform.

To overcome the difficulties of compacting a damp aggregate and to do it without breaking particles, the aggregate is inundated, followed by vibrating, jarring, or jolting with a predetermined amount of effort. A quantity of the aggregate is placed in a calibrated measure, and weighed. Water is then introduced, slowly, to avoid washing, from the bottom upward, to displace the air. With the aggregate barely covered with water, the measure is vibrated or jolted mechanically. This should be gentle, to avoid segregation, and mechanical, in order to make it reproducible. No definite degree of compaction is necessary except that it should be carried far enough to insure reproducibility. A disc and measuring rod is then placed on top the aggregate, by which means the

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volume of the compacted aggregate is obtained. A table can be prepared to facilitate this step. Thus the weight of aggregate placed in the measure and its volume are known, from which the weight per C.C.F. is calculated.

The measure should have at least 1/10 cu. ft. capacity, and the ratio of height to diameter should be of the order of 1-½. Introduction of the water is accomplished by a small tube entering the side of the measure close to the bottom. A water can having a nipple soldered in its side is connected to the measure by a 3-foot length of rubber tubing, so the flow is by gravity.

A cover for the measure may be needed to prevent splash, but depends on the compaction method. When the aggregate is placed in the measure, the scoop should be rotated to avoid segregation. By careful filling, striking the top off even with the measure, and avoiding rough shaking or jolting, a flat surface will result for the disc and measuring rod to rest on.

This is not precision, as said before, but a determination can be made in 10 minutes or less, and more frequent determinations may result in better control than slower methods. It can be seen that, while prewetting of the aggregate in the stock pile is desirable and good practice, it is not essential to controlling batch weights.

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Current Reviews

of Significant Contributions in Foreign and Domestic Publications, prepared by the Institute's Reviewers

Design of concrete pavements requiring a minimum of steel

HIGHWAY RESEARCH BOARD, September 1942; 13 p. (No. 3; of a series of bulletins on Wartime Road Problems.)

Steel, commonly used in concrete pavements, in bar mats or wire mesh, tie-bars across hinged longitudinal joints, including chairs and pins for support, deformed metal dividing plate for a longitudinal hinged joint and dowel bars or other load transfer devices. The bulletin discusses methods of reducing the use of distributed steel, elimination of the need for tie-bars and of steel dividing plates.

Displacements and stresses in the general two-dimensional framework

YVES NUBAR, Columbia University, 1942. Reviewed by S. LEVY

This paper describes a general method for the analysis of two-dimensional frameworks. The method takes account of both apial and lateral deformations of the members. Application of the method to problems such as deflection of trusses, pin-connected structures with redundant members and secondary stresses is outlined. A special type of solution is evolved for such frames as viaduct towers, Viervendeel trusses and other structures consisting of panels of polygonal shape. Several numerical examples are given.

Curing concrete pavements under wartime restrictions on critical materials

HIGHWAY RESEARCH BOARD, July 1942, 16-p. No. 1 of a series of bulletins on Wartime Road Problems.

Presents a "Recommended Practice for Curing Portland Cement Concrete Pavement under Wartime Restrictions on Critical Materials"—also suggested as "suitable for floors, walks and other like slabs". The recommendations are made with special reference to the unavailability of burlap, the restricted use of cotton mats and waterproof paper, the general unavailibility of bituminous materials in some regions and in recognition of the availability of other materials. The recommendations cover the use of wet coverings, membrane seal coats and waterproof paper and present current ATSM standards for the materials involved.

War plant buildings without footings

Engineering News-Record, V. 129, No. 11 (Sept. 10, 1942), pp. 84-86. Reviewed by S. J. CHAMBERLIN

The foundations are 6-in. plain concrete floor slabs, combined with wall and column supports, built on a raised compacted base of clay and crushed stone. The topsoil is excavated deep enough to remove all organic material and is replaced with selected

clay and compacted. Then two courses of crushed stone, crusher run maximum 4-in size are placed, each compacted to a 5-in. thickness. The top is leveled off with a compacted top course of 1-in. maximum size stone. Edges of the slab under heavy walls are thickened to 11 in., and under trucking aprons to 8 in.

More production from truck mixers

HAL. W. HUNT, *Engineering News-Record*, V. 129, No. 11 (Sept. 10, 1942), pp. 94-99.
Reviewed by S. J. CHAMBERLIN

Suggestions from manufacturers, distributors and operators on how to place more concrete by truck mixers are discussed. Transfer-hoppers, big buckets and better planning can speed unloading of trucks. Batching plants should provide easy access for the trucks, have sufficient capacity for full batches and utilize automatic weighing equipment. When conditions permit, it is desirable to measure the water at the plant and introduce it directly into the drum along with the aggregate and cement. Cleaning should be thorough to secure efficiency in mixing and discharging. The wash water should be dumped.

Planned formwork speeds construction

Engineering News-Record, V. 129, No. 9, (Aug. 27, 1942), pp. 66-68. Reviewed by S. J. CHAMBERLIN

Complete plans for forms and their erection and careful attention to details enable 100 carpenters to prepare column and deck forms for an average 25,000 sq. ft. of concrete slab daily on a multistory housing project. Plywood is used for all forms with each piece detailed for a particular location. The forms are designed to be handed up manually from one floor to the next. Long boom cranes with gooseneck jibs place concrete to the roof over the thirteenth floor without the use of runways and buggies. Single-line, bottom-dump buckets take the discharge from truck mixers. A 2-cu. yd. bucket is used with moderate length booms and a 1½-cu. yd. bucket for the large boom crane.

Air raid precautions

Indian Concrete Journal, May-June 1942. Reviewed by J. C. PEARSON

This two-month combined special issue is devoted almost entirely to protection against bombs from the air. It is a compilation of broad scope, endeavoring to embody the best of present day thought on the design and construction of private and community shelters. While the advantages of concrete construction are emphasized, the publishers have given relatively much more space to an enumeration of the factors that govern the choice of suitable protection, from the characteristics of bombs and their effects to detailed plans of all types of shelters. This issue is profusely illustrated not only with diagrams but many excellent photographs of different types of protection and bomb damage to notable buildings in England.

Stovepipe forms speed winter concrete

Engineering News-Record, Vol. 129, No. 11 (Sept. 10, 1942), pp. 80-81. Reviewed by S. J. CHAMBERLIN

Stovepipe sections were fitted together and the lower end inserted in a wooden footing form and nailed in place. The completed forms were placed as soon as possible after the excavation was complete and backfilling was begun immediately. Wooden braces were used in each pipe to prevent collapse. High-early-strength concrete of preheated materials was placed as quickly as possible. The completed pier was covered by a wooden barrel that also housed a kerosene lantern. When concrete was not added immediately, a kerosene lantern was suspended in each stovepipe from the board used to close the top. When protection against low temperatures was unnecessary, cylindrical cardboard forms were faster and more economical.

Modulus of elasticity of concrete exposed to vibrations

G. Eilers. *Beton u. Eisen*, 1941, 40 (20), 268-72; (21), 280-2. *Building Science Abstracts*, Vol. XV, (New Series), No. 3, March, 1942. Highway Research Abstracts

An account is given of an investigation of the bending moments in reinforced concrete beams exposed to vibrations with the object of determining the value of the modulus of elasticity (E_c) of the concrete to be considered in the design of structures under these conditions. From the results it was concluded that where only slight vibrational stresses are to be anticipated, i.e., in all cases where vibrations are as far as possible to be avoided, the value of E_c = 4,260,000 psi may be used. In other cases, it would be necessary to determine the magnitude of the stresses set up and to select accordingly a mean value of E_c obtained, if necessary, from statical tests. The most important result of the experiments is considered to be the conclusion that there is no difference in the elastic behaviour of concrete in respect to static loads and rapid vibrations, but that in each case the value of E_c bears the same relation to the stresses set up. The view that concrete and reinforced concrete show marked damping of vibrations is not confirmed.

Operating a 9-mile belt conveyor

A. H. Wittemore, *Engineering News-Record*, V. 129, No. 7, (Aug. 13, 1942) pp. 86-88.
Reviewed by S. J. Chamberlin

Almost 6,000,000 tons of aggregates have been delivered in nearly two years of successful operation by the 9.6-mile belt conveyor system that serves Shasta Dam (see "Effect of Belt Transportation on Concrete Aggregates by Gordon L. Williams, ACI Journal, Feb. 1942; *Proceedings* V. 38, p. 329). The average delivery rate has been 1,000 tons per hour of actual operating time. Minor additions have included a deflector plate to avoid over-loading and a chute to lower the aggregate stream onto the succeeding belt at each transfer point. Belt wear has been slight and has not varied with the slope. Slippage of belts during wet weather was overcome by attaching lagging, with a roughened surface, to the drive pulleys. Silt loss in transit exceeds fines made by grinding at transfer points. Some chipping and breakage occurs in transit, the cobbles and coarse gravel showing a difference of 3 per cent in undersize material from end to end. The increase in undersize in the two smaller sizes of gravel is negligible. Reduced volume increased breakage.

Handbook for Concrete and Cement

War Department, Corps of Engineers, U. S. Army, North Atlantic Division, Central Concrete Laboratory; 384 pages, 28 plates, 34 figures, 11 tables, October 1942, $3.00. Reviewed by Charles E. Wuerpel.

A handbook of field and laboratory test methods, inspection procedures and relevant specifications. Applicable Federal Specifications for Cements and Standards of the American Society for Testing Materials as well as special methods developed at the Central Concrete Laboratory are included. The rationalization of methods and presentation of theoretical background is not attempted since the volume is simply a working tool and a ready source for reference. The handbook was prepared by authority of the Chief of Engineers, U. S. Army for distribution by sale within the Department and to the public. It should prove of interest and value to office engineers, laboratory workers, and field inspectors concerned with testing and inspection of, or research on, concrete and its ingredients.

Modern concrete compaction methods

Niels M. Plum, Engineering Firm of Christiani and Nielson, Copenhagen, Denmark. A lecture delivered to the Danish Engineering Society, Feb. 6, 1942. Reviewed by P. von der Lippe

A discussion of the various concrete compaction methods, with particular attention to vibration, in which the author compares the advantages of superior compaction

methods to those which require concretes of greater cement and fine aggregate content. The newer methods described include: spinning, as used for concrete pipes and poles; vacuum process; jolt concrete, such as employed for precast members; machine tamping of highway slabs; and vibration. Discussion of the latter covers: an analysis of the forces set up by vibration and shows how vibration overcomes internal friction of the aggregate particles to produce flow and secure their compaction; the importance of frequency, acceleration and amplitude; and the amount of vibration. Among the advantages of vibrated concrete the author cites: economy in cement; reduction of pore space; improved tensile strength, compressive strength, bond, density; and improved resistance to weather, wear, chemical attack and frost action. Practical hints are given on vibration for building construction, mass concrete, road work and precast concrete. Proper methods for handling concrete and aggregates are explained, with illustrations of American practices.

Concrete Work

E. Molloy, 140 p. illus., Chemical Publishing Co., Inc., Brooklyn, $2.50. Reviewed by J. W. Kelly

Practical books of this kind are commendable in that they are close to the small job and are addressed primarily to builders and foremen, for whom the bulk of technical literature has no appeal. The importance of workmanship is emphasized by such statements as "it is possible for the (given) concrete to vary in strength by as much as 50 per cent when made by different gangs". The eight chapters include: I. Concrete Materials, Mixing and Placing; II. Steel Reinforcement, Bending and Fixing; III. Concrete Foundations, Walls, Floors, and Roofs; IV. Formwork for Concrete Buildings; V. Precast Work; VI. Concrete Paths and Garden Steps; VII. Concrete Roads and Road Repairs; and VIII. Surface Treatment of Concrete Walls and Floors. While many terms and details of English practice as described in this book differ from those in America, the principles do not; and in the main the American builder will profit from English experience. The book is sound in recommending the avoidance of extremes in mixes—wet or dry, coarse or fine—rich or lean. One interesting recommendation is that the water content of the mix be fixed by supplying say 28 per cent by weight of the cement plus say 4 per cent by weight of the aggregate. (Incidentally, an English bag of cement contains 112 lb. and the Imperial gallon of 10 lb. is used.) The section on roads would be applicable here to relatively small paved areas where hand-finishing methods are used. The book is well illustrated, and gives specific examples of typical members such as footings, lintels, precast ornamental units, and formwork. It is one of a "Building Practice" series which includes also books on Roof Construction and Repair, Brickwork and Masonry, and Builder's Machinery and Equipment and a Plant Engineer's Manual.

New Zealand emergency standard code for raid shelter

New Zealand Standards Institute, War Emergency Supplement 1, March 1942.
Reviewed by D. E. Parsons

Supplement No. 1 is divided into two parts, the first consisting of a review and discussion of the code for raid shelters, and the second of notes on the protection of window openings. Since the publication of the code for raid shelter, new information on requirements for and the performance of raid shelters has been obtained principally from Great Britain. Because of the scarcity of steel reinforcement, information had been sought as to the feasibility of substituting timber bracing for reinforcement in walls of brick masonry. A review of existing data indicates that designs may be considered adequate for resisting blast pressures if based upon a pressure of 5 lbs. per sq. in. for panels up to 50 sq. ft. in area, 3 lbs. per sq. in. for panels 50 to 100 sq. ft.

in area, and 100 lbs. per sq. in. for panels of greater area. Roofs should be firmly attached to walls. Reinforced concrete floors properly designed for live loads of 80 lbs. per sq. ft. ordinarily have sufficient strength to be a satisfactory protection against falling debris. In any case, designs should be based upon ultimate loads rather than yield loads. Space requirements for children may be reduced to between ½ to ⅔ of those for adults, provided the space is adequately ventilated. Large shelters should be subdivided into smaller units with a capacity of not more than 50 persons for each. It is advisable to provide connections between shelters where that is feasible. Drainage systems preferably should not be attached to sewers.

Part 2 describes and illustrates methods for protecting window openings. Blocking is considered the easiest and cheapest way of affording protection in ground floor window openings. Types of blocking include concrete blocks, wooden frames filled with crushed stone, brick masonry, and sand-bag barricades. Details of timber shelters capable of withstanding blast are given.

Possibilities of improvement of the quality of concrete by an admixture

DR. VON MENG, *Zement*, Apr. 16, 1942 (from translation provided by Sika, Inc.)
Abstract by F. B. HORNIBROOK

The author discusses the factors affecting "pump-crete" and tests made by French testing laboratories in occupied France of a plasticizing material, Plastiment, developed in Switzerland and said to have been used considerably in construction in "occupied territories".

For reinforced concrete having maximum size of aggregate of 1¼ in. to be pumped through a pipe 6 in. in diameter it is stated that 1¼ size should not constitute more than 15 per cent of the total aggregate; the sand passing the No. 3 sieve (U. S. Standard) should be about 50 per cent and not over 60 per cent of the total. At least 5 per cent of the sand should pass the No. 70 sieve. The sizes .3 to .6 in. should be present in about 15 per cent and there should be 20 to 25 per cent of the .6- to 1.2-in. size. Under normal conditions approximately 6 to 8 per cent of water by weight of the total concrete should be used, resulting in a slump of 2½ to 3½ in. A minimum of 4½ bags of cement per cubic yard of concrete is recommended.

In laboratory tests of concrete obtained from a construction job, with an 8 bag mix the water-cement ratio was reduced from .58 to .52 for a slump of approximately 3½ in., and the 7 and 28 day average compression strengths of cubes were increased 42 per cent and 28 per cent respectively, by the addition of one per cent (by weight of cement) of Plastiment. In a series of tests made at a different laboratory it was found that both tensile and compressive strengths were somewhat greater for a 6.28 bag mix containing Plastiment than was obtained with a 7.17 bag mix without admixture, all mixes having approximately 3 in. slump. It is stated that the addition of Plastiment improves the workability and enables increases of 30 to 35 per cent in the 7 and 28 day compressive strengths (by taking advantage of the lower water requirement).

It is emphasized that these results hold only for standard portland cements, also that the initial and final set is delayed several hours as well as the development of strength up to 3 days. Amounts of Plastiment greater than one per cent are apt to affect unduly the setting and early strength.

Mechanical finishing of concrete in road construction with special reference to vibration: A new way of increasing output

R. SCHADE: Berlin, 1940 (*Zementverlag* G.m.b.H.), RM. 6; Z. Ver. dtsch. Ing., 1941, 85 (32), 691-2. *Road Abstracts*, Vol. IX, No. 6, May 26, 1942. HIGHWAY RESEARCH ABSTRACTS

The suitability of various types of compacting and finishing machinery for the continuous production of single-course concrete road surfacings (i.e., the levelling, com-

paction and finishing of the concrete in a single traverse of the machine) has recently been studied at the Technische Hochschule Berlin, under the auspices of the German Research Association (Deutsche Forschungsgemeinschaft) and the Armaments Department of the F.M.B. (Forschungsinstitut fur Maschinenwesen beim Baubetrieb) or Institute for Research in Mechanical Engineering as applied to Construction Work. To insure strictly comparable conditions, the experiments were conducted indoors on test slabs 5 ft. wide and 8 or 10 ft. long, with a compacted thickness of 8.7 in. The preparation of test beams facilitated by vibrating strips of metal sheet into the freshly compacted concrete, dividing it into transverse strips 16 to 20 in. wide. The bending strengths of beams and the compressive strengths of test cubes were determined at 28 days. The machines used were: (a) a vibro-finisher, which compacted and finished the concrete by means of a vibrating screed; (b) a heavy tamping board working in the rear of a levelling board, the surface finish being imparted by a separate machine carrying a vibrating screed; (c) a finishing screed with a transverse reciprocating motion, mounted behind a levelling board and a tamping unit which were not used in this investigation. The water-cement ratios of the concrete were 0.42 and 0.45 for machines (a) and (b) and 0.5 and 0.525 for (c). The influence of the following factors on the finished concrete was studied; the water-cement ratio; rate of advance of the machine; frequency of impulse (from 1,560 to 6,840 per min.), or number of blows per meter length of road. The tests were carried out on concretes made with different cements and having different consistencies. The main results are summarized as follows: With the vibro-finisher (earth-moist concrete), higher frequencies produced a greater depth of compaction, and higher bending strength; lower frequencies produced lower strength, but better surface finish, and the tendency to form corrugations was less. Increasing the rate of advance of the machine decreased the depth of compaction and the bending strength of the concrete. Strengths obtained from concrete with the heavy tamping machine (earth-moist concrete) working at 320 blows per min. were about the same as those obtained with the vibro-finisher at 6,000 impulses per minute. A smaller number of blows yielded poor results and the degree of compaction decreased correspondingly. The necessity is emphasized for a special finishing screed or vibro-finisher to follow the tamping board to obtain the necessary surface finish. The tests with the finishing screed (wet concrete) yielded low bending strengths (about 640 lb. per sq. in.); the degree of compaction was sufficient only when a vibro-finisher was used. The surface finish was good throughout, but again the vibro-finisher was superior to the finishing screed. The results confirm the earlier conclusion that single-course concrete must be completed in a single, continuous operation. For this purpose a finisher is required that possesses two vibrating units: the compacting unit should work at a speed of 4,200 impulses, and the finishing unit at 2,100 impulses, per minute. The best rate of advance for the machine is between 1.3 and 2.6 ft. per min.; the speed shall not exceed 3.3 ft. per min. for earth-moist concrete (water-cement ratio 0.43), and 6.6 ft. per min. for wet concrete (water-cement ratio 0.49).

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The Design and Construction of the Concrete Multiple Arch Bridge over the Spillway of Grand Coulee Dam*

By E. R. DEXTER†

SYNOPSIS

This paper gives a general description of the design and construction of this reinforced concrete multiple arch highway bridge over the spillway of Grand Coulee Dam on the Columbia Basin Project, Washington.

INTRODUCTION

The Grand Coulee Dam spillway bridge is a reinforced concrete multiple arch structure to provide a roadway over the spillway portion of the dam. It was designed in the Denver Office of the Bureau of Reclamation, and was constructed under the supervision of this Bureau by the Consolidated Builders, Incorporated, as a part of their contract for the completion of Grand Coulee Dam. Construction of the bridge was started in July 1941, and it was completed in November 1941, except for the guard railings. Due to the national defense preparations it was impossible to secure the aluminum necessary for construction of the railings and in place thereof temporary wood railings have been erected. The bridge has never been opened to public traffic and will be kept closed to such until after the war, and until it will be possible to construct the permanent guard railings.

Fig. 1 shows a view of the bridge from the downstream side; Fig. 2, the bridge nearing completion, with centering in place for all of the arches, and also one of the construction cranes which operated on the trestle projecting through the downstream face of the dam. The concrete mixing plant was located at the end of the trestle on the right abutment, looking downstream.

The bridge has an over all length of 1650 ft. and consists of 11 units supported by piers 15 ft. thick in the direction of traffic and spaced at

*Received by the Institute June 29, 1942.
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Fig. 1—Completed bridge from down-stream side

150-ft. centers. Each unit consists of a roadway section supported by vertical cross-walls resting on a barrel type parabolic arch.

CONSTRUCTION OPERATIONS

Contract specifications required individual centering for all arches so as to prevent unbalanced loading of the piers. Each arch centering unit consisted of a deck system supported by 15 three-post bents resting on the spillway crest, partly on the concrete and partly on the spillway gates in their lowered position. To keep the deck adjusted to a theoretical curve, 12 sets of hardwood wedges were inserted between the cap of

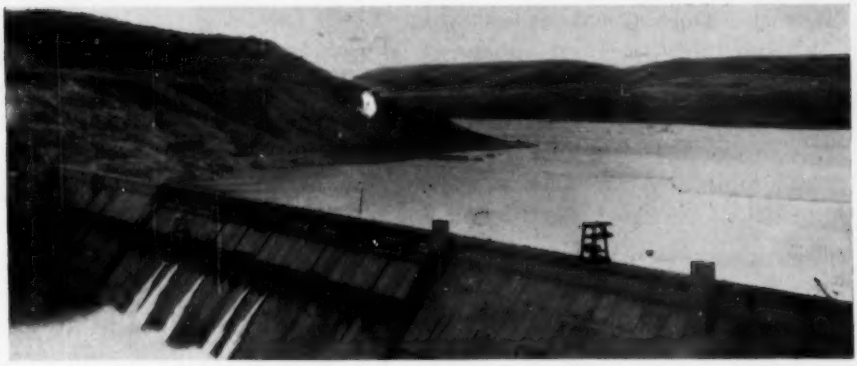


Fig. 2—View of bridge from downstream during construction (Nov. 4, 1941)

each bent and the joist sill above it. In general the centering differed very little from that used for many other arch bridges. The specifications also required that the centering be left in place for at least 30 days after all concrete had been placed in the arches. No construction of the superstructure was permitted until the arches were allowed to take their own weight by simultaneously lowering the centering of all arches for uniform loading of the piers. To minimize unbalanced loads on the centering during construction of the arches and on the arches during construction of the superstructure it was necessary to construct each unit in segments in the order shown in Fig. 3. Accordingly, the concrete was placed in corresponding segments of all units as nearly at the same time as was feasible and practical.

The contractor was required also to place the concrete in the arches under certain temperature conditions; except, that concrete could be placed in segments 1 and 2 as soon as the forms were ready. Concrete was permitted to be placed in the number 3 segments if the temperature of the concrete in segments 1 and 2 was down to 70F. or less, and it was permitted to be placed in the arch closure segments, numbers 4, if the concrete temperature in arch segments 1, 2, and 3 was down to 70F. or less. Temperature readings of previously placed concrete were secured from thermometers placed in wells in the bosses for the cross-walls. Cooling of the concrete was most effectively accomplished by fog spraying with river water.

DESIGN CONTROL

The design of the bridge was made in accordance with the following regulations:

Specifications pertaining to design loadings

1. Standard Specifications for Highway Bridges, adopted by the American Association of State Highway Officials, Second Edition—1935, and Third Edition—1941.

2. Design Specifications for Highway Bridges and Incidental Structures adopted by the State of Washington, Department of Highways—June 1935.

Allowable working stresses

1. Recommended Practice and Standard Specifications for Concrete and Reinforced Concrete, Section 878; Report of the Joint Committee, June 1940.

CAMBER

Each bridge span was designed to have a one-inch camber from the pier to the center of the span at a mean temperature of 50F. If, at the time of construction, the mean temperature varied from 50F., the elevations for the roadway were to be determined by applying a correction ordinate of elevation. This correction was to be calculated from a parabolic distribution in which the major axis would be equal to the length of the span, and the half minor axis would equal 1¼ in. for a 50F. rise or fall from the 50F. mean temperature. The temperature of the structure would be assumed to be equal to the mean air temperature of the three preceding days.

DESIGN LOADINGS

Dead load

This load includes, in addition to the weight of the component parts, an allowance of 15 lbs. per sq. ft. of roadway for a future wearing surface. The average unit weight of concrete including the weight of the reinforcement was assumed to be 160 lbs. per cubic foot.

Live loads

Included in this classification are loadings caused by highway traffic, impact, and pedestrian movement on the sidewalks.

1. Highway traffic. Two types of moving loads were considered in this group. These were the standard H20 and the H20-S16 design loadings adopted by the American Association of State Highway Officials.

2. Impact. An allowance of 30 per cent of the live load effects was used to compensate for dynamic, vibratory, and impact disturbances of traffic movement for design of the arches. The amount of impact used for the design of the roadway slab was determined from the formula $I = \frac{50}{L + 125}$, in which I is the impact fraction and L is the design span length.

3. Sidewalk live loads. Pedestrian traffic was assumed to require an allowance of 100 lb. per sq. ft. of sidewalk area for the design of the cantilevered sidewalk slab. In addition to this load a railing load equivalent to 150 lb. per lin. ft. of railing applied at the top of the railing was assumed. An allowance of 60 lb. per sq. ft. of sidewalk area was made for the sidewalk effect on the arch barrel.

Temperature changes

Design considerations included the possible effects of temperature variations from an assumed closing temperature of the arches. For the purpose of making allowances for possible fluctuations of stress distribution in the arches it was assumed that the closing temperature would be 70F. Accordingly, the reinforcement was designed to accommodate a drop in mean concrete temperature of 70F. and a rise of 45F. Actually, the closing temperature of one of the arches was 77F. and the calculation of stresses for record purposes was based on this closing temperature.

Rib shortening

Because of the low rise-span ratio (0.122) of the arches of this bridge, the effects of rib shortening were included in all stress and deflection calculations.

Shrinkage

To compensate for the stresses induced in the arches due to shrinkage caused by the drying and curing of the concrete, an allowance equivalent to a 15F. temperature drop was made.

Special loads

Besides the preceding ordinary design loads it was assumed that the bridge would be subjected to the following loads; (1) earthquake shocks, (2) differential water load on a pier, and (3) heating of pier plates for de-icing purposes. Discussion of these loads is given later in this paper.

PIERS

Pier design did not enter into the problems encountered in the design of the bridge. Due to the use of drum gates for the spillway control, the necessary piers between gates provided ideal supports for the bridge spans. Although a floatwell and stairwell were located in each pier, there is sufficient reinforcement embedded near the surfaces of the piers and around each well to make the piers fully capable of supporting the bridge for any non-symmetrical loading due to traffic movement. The reinforcement is well embedded in the mass of the dam and serves to provide lateral stability for the bridge in the case of a horizontal earthquake shock.

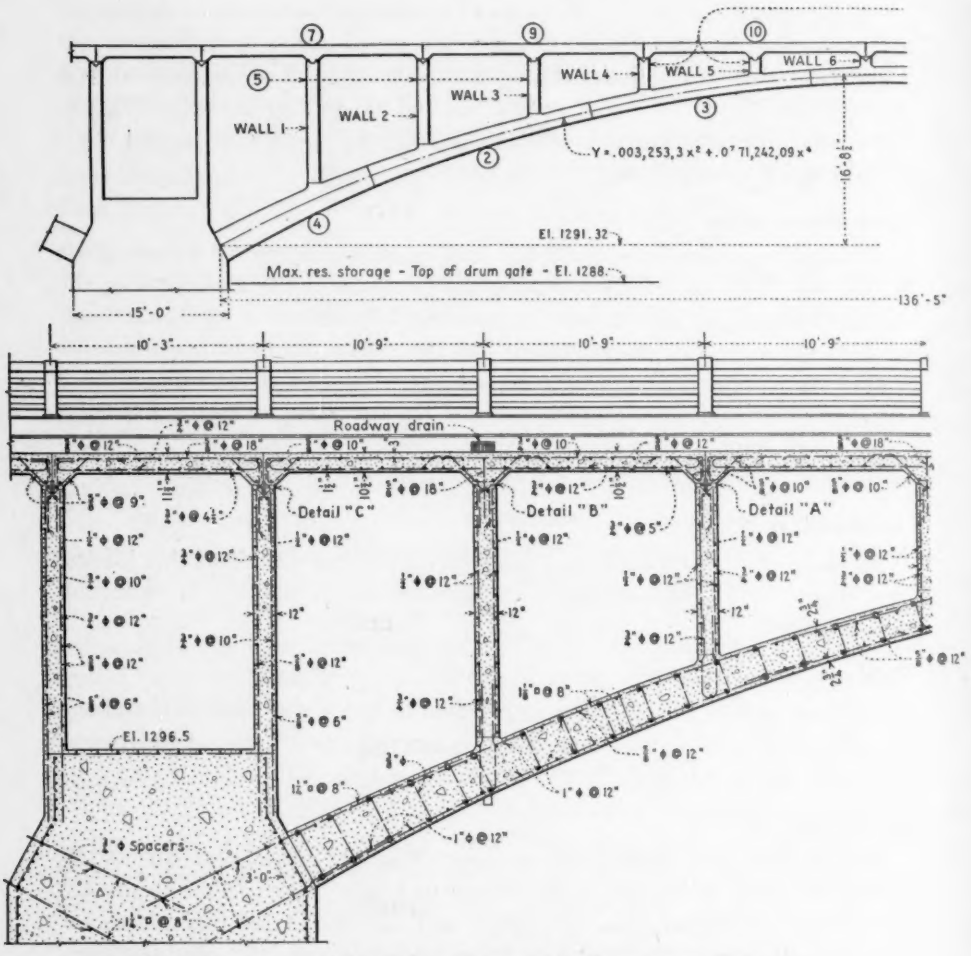


Fig. 3 (across top of this and opposite page)—Construction sequence

Fig. 4 (across this and opposite page)—Half span section at centerline

DESIGN OF ARCH BARREL

Determination of arch shape

The shape or curvature of the arch barrel was determined by use of the equations developed by Victor H. Cochrane.* No variation from this shape was necessary after detailed stress analyses had been made. The realtively short vertical distance between the maximum water surface and the roadway surface, plus the desirability of having both top and bottom surfaces of the entire barrel exposed to the air for uniformity

*Proc. Engr. Soc. West. Penn., V. 32, 1916.

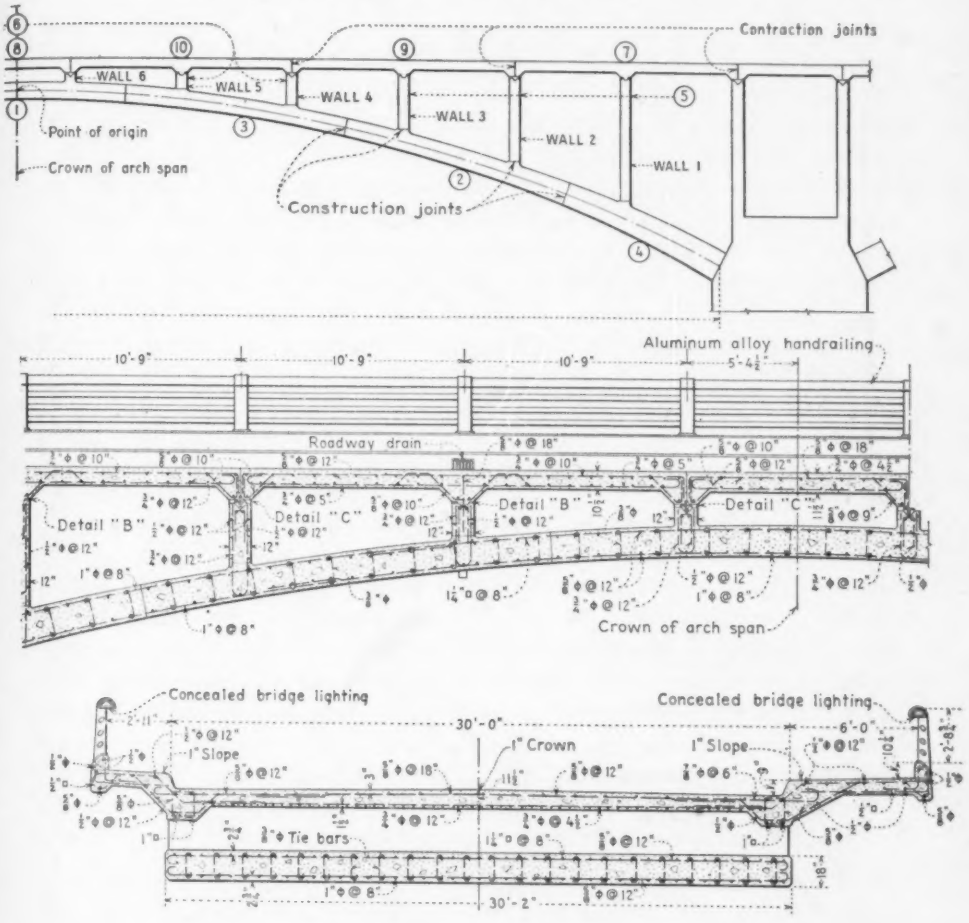


Fig. 5—Section at crown of arch span

of effects of temperature changes, resulted in a rise-span ratio of only 0.122 and a relatively small crown thickness.

Each arch barrel supporting the roadway has a width of 30 ft. 2 in., a clear span of 135 ft., and a rise of 16.68 ft. The thickness varies from 18 in. at the crown to 36 in. at the springing line.

Because of the position of the bridge at the top of the maximum section of the dam (Fig. 6) at about 550 ft. above the rock foundation surface it was considered advisable to construct the arches as rigid barrel segments rather than as arched ribs in order to afford greater lateral rigidity against earthquake shocks.

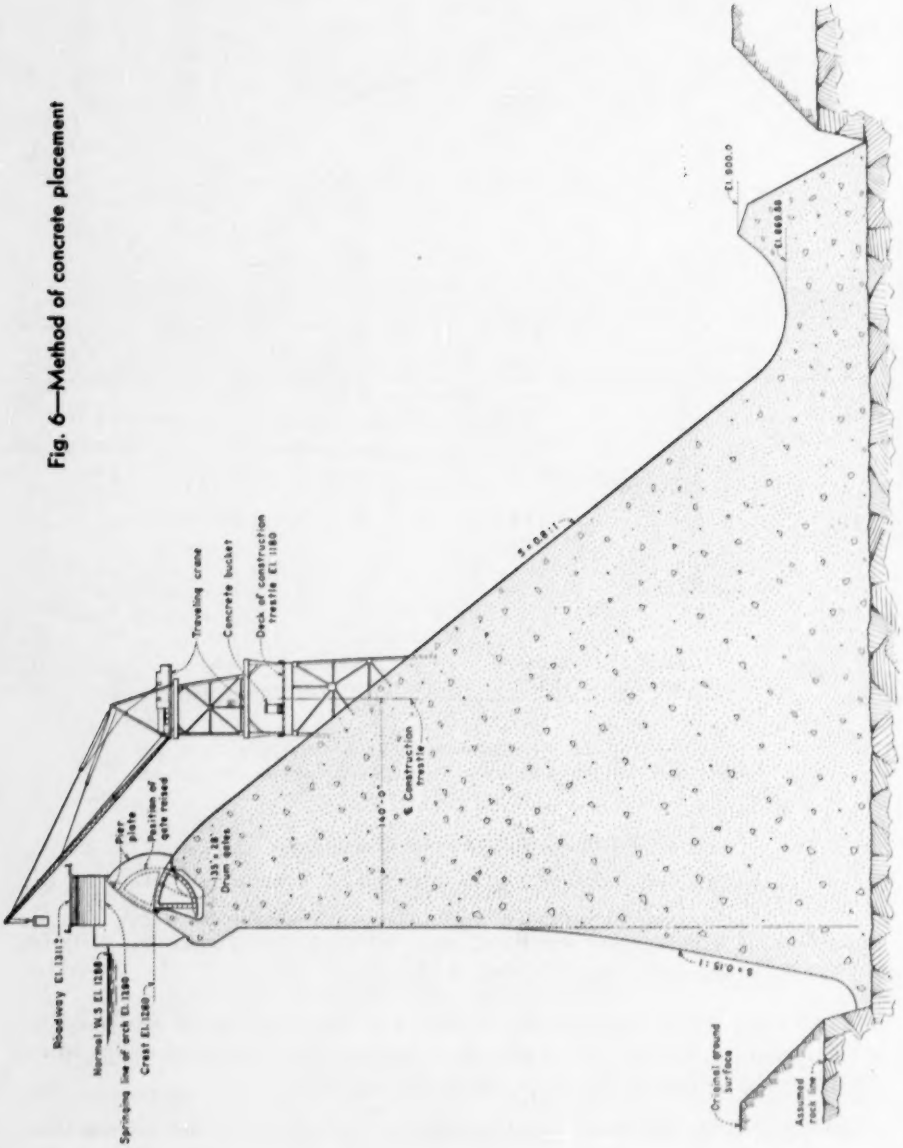


Fig. 6—Method of concrete placement

Arch reinforcement

The arch reinforcement consists of longitudinal bars at 8-in. centers, both faces, and transverse bars at 12-in. centers, both faces (Fig. 4). The two layers of steel are tied together with a ⅜-in. round tie bar for every 1⅓ sq. ft. of area. This spacing was not an arbitrarily chosen unit but rather the result of adherence to "Suggested Specifications for Concrete Arch Design."* The longitudinal steel in the extrados is 1¼-in. square bars at both crown and abutment, varying to 1-in. round bars near the quarter point of the arch. The intrados longitudinal steel consists of 1-in. round bars from the crown to about the haunch where 1⅛-in. square bars are used. The steel at the abutment intrados is 1¼-in. square. The transverse steel consists of ⅝-in. round bars in both faces for the entire arch except near the abutments where 1-in. round bats are used.

Arch stresses due to normal loads

Moments, thrusts, and shears in an arch, for the several loading conditions, were calculated by using the method presented in the United States Department of Agriculture publication† of August 1927, entitled, "The Analysis of Concrete Arches," by W. P. Linton, Highway Bridge Engineer and C. D. Geisler, Associate Highway Bridge Engineer, Bureau of Public Roads. Unit load moments, thrusts, and shears were computed for ten points on each half of the arch, and from these results influence curves were prepared for unit loads applied at each wall point. All live loads and the weight of the superstructure are transferred to the arch at the cross-walls in the form of concentrated loads, whereas the weight of the arch and temperature change loads are distributed along the arch.

Stresses were computed for load combinations including dead load, live load (highway and sidewalk), 30 per cent impact, concrete shrinkage, and temperature changes. A tabulation of these stresses is given in Table 1. An examination of this tabulation will disclose a maximum tensile stress of 18,890 psi in the steel and a maximum compressive stress of 1356 psi in the concrete. Allowable working stresses for design were 20,000 psi tension in the steel and $0.45 f_c$ psi compression in the concrete. For design purposes f_c was assumed as 3000 psi, making the limiting unit compressive stress in the concrete equal to 1350 psi. During the construction of the bridge, 152 samples of concrete were taken for 6x12-in. cylinder tests. This concrete also included that required for construction of other features of the dam using the same mix, and out of this total number of samples tested at the age of 28 days the lowest single test indicated a compressive strength of 3710 psi. This value would have

*"Plain and Reinforced Concrete Arches," Report of Committee 312, ACI JOURNAL, Sept., 1940, *Proc.* V. 37, 1941, p. 1.

†Reprinted from *Public Roads*, V. 8, No. 4 and 5, June and July 1927.

TABLE 1—ARCH STRESSES—NORMAL LOADING CONDITIONS

Arch Points		Bridge Loading	
		Dead Load—H20 Live Load—Sidewalk Live Load 30 per cent Impact	
		Concrete Shrinkage 77F. Temp. Drop	45F. Temp. Rise
		Stresses in pounds per square inch.*	
Abutment	Extrados	−18,890	+568
	Intrados	+ 1342	−971
Wall 1	Extrados	−12,455	+679
	Intrados	+ 1238	−794
Wall 2	Extrados	− 3930	+600
	Intrados	+ 977	+ 30
Wall 3	Extrados	+ 39	+798
	Intrados	+ 645	0
Wall 4	Extrados	+ 977	+ 99
	Intrados	− 2745	+679
Wall 5	Extrados	+ 1349	+ 87
	Intrados	−10,315	+633
Wall 6	Extrados	+ 1356	+ 69
	Intrados	− 9840	+635
Crown	Extrados	+ 1111	+ 72
	Intrados	− 5245	+626

*Plus sign (+) denotes compression in concrete.
Minus sign (—) denotes tension in arch reinforcement steel.

permitted, for design, a working stress equivalent to 0.45 times 3710 or 1670 psi. The maximum compressive strength at age 28 days was 7110 psi and the average of all tests was 5317 psi. Although these tests indicate an uneconomical use of cement, it is believed that the more durable concrete thus obtained more than justifies the extra cost.

Arch stresses due to special loads

In addition to the stresses listed in Table 1 stresses were computed for studying the effects of (1) an H20-S16 highway loading, (2) earthquake shocks, (3) differential waterload on a pier, and (4) heating of pier plates for de-icing purposes. A brief explanation of these loads is given for general information purposes.

1. An H20-S16 load is a relatively new highway traffic design load, but its use is not mandatory. It consists of a 16-ton semi-trailer coupled to an H20 truck, making a total wheel base length of 28 ft.

2. Earthquake shocks were assumed to produce an effect equivalent to an acceleration of 0.1 times gravity acting upon the mass of the bridge,

both in a vertical direction and in a horizontal direction parallel to the line of traffic.

3. A differential water load is a loading condition incidental to the operation of the spillway gates for the regulation of the reservoir. This condition will arise when the reservoir storage is at a maximum and one gate is completely lowered, thereby causing a difference in water pressure on the sides of the piers adjacent to the lowered gate. The unbalanced loading thus created will cause the piers to deflect and disturb the stress distribution throughout the arches.

4. Stress concentrations caused by heating the pier plates is a condition that might arise during winter operation of the spillway gates. The pier plates are constructed of segments of cast iron embedded in the sides of the piers at the ends of the spillway gates in their raised position (Fig. 6) and serve to provide a durable, plane surface at the ends of the gates for sealing purposes. A stress disturbance in the arches will arise if the piers are deflected due to heating these pier plates for de-icing the ends of a gate so that it may be lowered. For the purpose of a stress investigation it was assumed that, with water in the reservoir to the top of the gates and with one or more gates open, a severe loading could develop on the piers between the open and closed bays if it were necessary to heat the pier plates at the ends of the raised gates. This assumption combines the effects of pier plate heating with the effects of a differential water load on the pier to produce a severe stress concentration in the arch barrels supported by the pier.

Table 2 lists the maximum stresses that were computed for these out-of-the-ordinary conditions. In all of the investigations for maximum stress possibilities, it was assumed that these special loadings could occur at times of maximum highway traffic and greatest temperature change. Maximum combined stresses for other than normal loadings are permitted to exceed the allowable by one-third. For this bridge with the quality of concrete specified and with the use of intermediate grade steel the maximum stresses would be allowed to approach $1\frac{1}{3}$ x 0.45 x 3000 or 1800 psi compression for the concrete and $1\frac{1}{3}$ x 20,000 or 26,670 psi tension in the steel.

DESIGN OF SUPERSTRUCTURE

The superstructure of the bridge is composed of all component parts above the arch barrel and consists of the roadway section, the sidewalks, the railings, and the supporting cross-walls between the deck and the arch.

Roadway section

The roadway deck slab is 30 ft. wide between curbs and has a thickness along the centerline of $10\frac{1}{2}$ in. for the two-span continuous segments and

TABLE 2—ARCH STRESSES—MAXIMUM LOADING CONDITIONS

Loading	Maximum Stresses Pounds per square inch	
Dead Load, Concrete Shrinkage 77F. Temp. Drop, and Sidewalk Live Load Plus	Compression in Concrete	Tension in Steel
H20-S16 traffic load + impact	1559	22,140
Earthquake shock H20 traffic load + impact	1752	22,450
Differential water load H20 traffic load + impact	1517	22,800
Heating pier plates Differential water load H20 traffic load + impact	1568	24,100

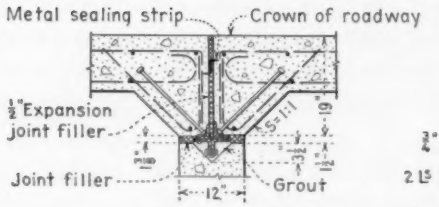
Note: Impact = 30 per cent traffic load.

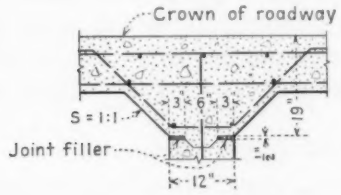
11½ in. for the simple spans at the pier and central segments (Fig. 4). A one-inch crown is provided for drainage. Besides being supported at the cross-walls the slab is also supported by a reinforced concrete spandrel beam at its upstream and downstream edges (Fig. 5). These spandrel beams also serve to support the cantilevered sidewalks that flank the bridge on each side. The sidewalk on the upstream side is 2 ft. 1' in. wide, and on the downstream side the walk is 6 ft. wide. Along the edge of each sidewalk will be a horizontal bar guard railing, the top bar of which will house an indirect lighting system. The railing is designed to provide maximum visibility for occupants of traffic vehicles and yet be consistent with the strength required to withstand shocks of uncontrolled vehicles. Details are shown in Fig. 4 and 5.

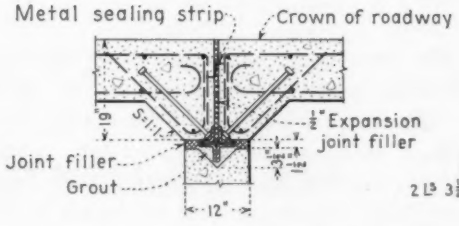
Jointing

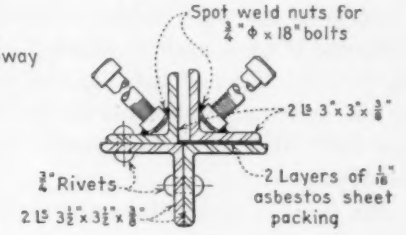
As a means to minimize the effects of contraction and expansion due to temperature fluctuations and to minimize the effects of strains due to arch flexure, the deck slab was articulated into longitudinal lengths of 10.75 and 21.50 ft. by transverse joints. This jointing also serves to prevent increased concentrations of load on the arches due to the same causes. Joints were provided for three different sets of conditions.

Fig 7 and 7a illustrate a condition wherein one end of a deck slab segment is rigidly fixed to its support and the adjoining end of an adjacent segment is free to slide. This type of joint was devised to relieve a horizontal thrust that might develop against cross-wall No. 1, Fig. 3 and 4. Otherwise it would have been necessary to make the wall thicker in order to keep the stresses within working limits. The bottom bearing element of the joint consists of two angles 3½ x 3½ x ⅜ in., riveted together and









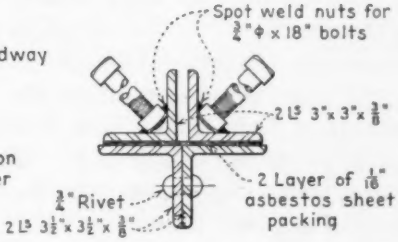


Fig. 7 and 7a (above)—Respectively detail "A" (Fig. 4) and bearing assembly for detail "A".

Fig. 8 (left)—Detail "B" (Fig. 4)

Fig. 9 and 9a (below)—Respectively, detail "C" (Fig. 4) and bearing assembly for detail "C".

grouted into the supporting cross-wall. The top bearing elements of the joint consist of two angles, 3 x 3 x ⅜ in., one of which is riveted to the bottom bearing element and the other separated from the bottom bearing element by two 1/16-in. sheets of asbestos packing. One of each of the upper bearing angles is anchored to one of the adjoining slabs by ¾-in. diameter bolts 18 in. long which were screwed into square nuts welded to the angle iron at 18-in. centers and then embedded in the concrete of the slab.

Fig. 8 illustrates a joint between a two-span continuous deck segment and its center support. This joint was made by forming a recess along the center line of the top of the supporting crosswall, and then keying the deck concrete into it. The concrete adjacent to the sides of the recess was separated from the deck by ½-in. thick strips of joint filler. This joint was designed to permit some rocker action, but no horizontal movement other than that due to the flexibility of the support.

Fig. 9 and 9a show a type of joint designed to permit horizontal movement but no rotation. In this joint the lower bearing plate is built of two angles, 3½ x 3½ x ⅜ in., riveted together and grouted into the supporting cross-wall. The upper bearing plates consist of two angles

3 x 3 x ⅜ in., each anchored to the adjoining ends of adjacent deck segments. Anchorage is effected by ¾-in. diameter bolts 18 in. long which were screwed into square nuts welded to the angles at 18-in. centers and then embedded in the deck concrete. The upper plates are separated from the lower plates by two layers of 1/16-in. asbestos sheet packing.

The joint between the ends of the deck slab segments is ½ in. wide and is filled with an expansion joint filler. Across this joint about 6 in. below the surface there is placed a metal sealing strip, welded to the reinforcement. This sealing strip was used to minimize moisture penetration to the angles in the joint. Each joint is continuous across the deck slab and both sidewalks.

Roadway slab

The roadway slab was designed with the main reinforcement parallel to the direction of traffic. An H20 truck train loading plus an allowance for impact was used for design.

The main reinforcement in the two-span continuous slab segments consists of ¾-in. round bars at 5-in. centers placed 1½ in. above the lower face of the slab (Fig. 4). The steel in the top of the slab parallel to the traffic movement consists of ¾-in. round bars at 10-in. centers placed 3 in. below the surface at the center of the roadway and 2 in. below the surface at the curbs. The reinforcement transverse to the main steel consists of ¾-in. round bars at 12-in. centers in the bottom of the slab and ⅝-in. round bars at 12-in. centers in the top of the slab. The maximum stresses in the two-span continuous slabs were calculated to be 16,820 psi tension in the steel and 940 psi compression in the concrete.

The reinforcement in the simple span roadway slabs, central and pier segments, consists of ¾-in. round bars at 4½-in. centers placed 1½ in. above the bottom surface of the slab. The reinforcement in the top of the slab parallel to traffic movement consists of ⅝-in. round bars at 18-in. centers placed 3 in. below the surface at the center of the roadway and 2 in. below the surface at the curbs. The reinforcement transverse to this steel consists of ¾-in. round bars at 12-in. centers in the bottom of the slab and ⅝-in. round bars at 12-in. centers in the top of the slab. The maximum stresses in these simple span central and pier segments was calculated to be 17,920 psi tension in the steel, and 1050 psi compression in the concrete.

Sidewalks

The sidewalks were designed as cantilevers fixed to the edges of the roadway slab and to spandrel beams constructed monolithically with the edges of the slab (Fig. 5). In addition to their weight and the weight of the guard railing, they were designed to carry a uniform live load of

100 lb. per sq. ft. plus a horizontal live load of 150 lb. per lin. ft. of railing acting laterally against the top of the railing. The stresses in the cantilevered sidewalk at its support were calculated to be 13,680 psi tension in the steel and 465 psi compression in the concrete.

Cross-wall supports

Dimensions of the cross-walls were determined arbitrarily, based somewhat on architectural considerations and minimum dimension requirements. Stresses were a factor in their design only for the first cross-wall (wall No. 1) next to the abutment. This situation was relieved by devising a joint of the type shown in Fig. 7 and 7a and using it at the top of wall No. 2 to relieve transfer of horizontal thrust to wall No. 1.

TABLE 3—DEFLECTION AT ARCH CROWN

Deflections due to	
Shrinkage (equivalent to 15F. temp. drop)	−0.028 feet
45F. temperature rise	+0.084
77F. temperature drop	−0.144
Weight of arch barrel & cross-walls	−0.016
Weight of roadway slab and sidewalks	−0.016
H20 traffic loading	−0.022
Sidewalk live load	−0.001

Minus sign (—) denotes downward deflection.
Plus sign (+) denotes upward deflection.

DEFLECTIONS

Table 3 presents a list of theoretical deflections at the crown of the arch calculated by the voussoir method of arch analysis.* Deflections were calculated for the effects of temperature changes, dead loads, live loads and shrinkage (assumed equivalent to a 15F. temperature drop). All calculations included the effects of rib-shortening and shear. No attempt is made to theorize what maximum deflection should be expected as it has not yet been reported by the field office to exactly what elevations the roadway surface was cambered.

*"Analysis of Arch Dams by the Trial Load Method," C. H. Howell and A. C. Jaquith; A.S.C.E. Trans., V. 93, 1929.

Discussion of this paper should reach the ACI Secretary in triplicate by April 1, 1943, for publication in the JOURNAL for June, 1943

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Discussion of a paper by E. R. Dexter:

The Design and Construction of the Concrete Multiple Arch Bridge over the Spillway of Grand Coulee Dam*

By HOMER M. HADLEY

AUTHOR'S ACKNOWLEDGMENTS, SUPPLEMENTING ORIGINAL PAPER

The author wishes to acknowledge the contributions that others in the Bureau of Reclamation designing office have made toward the design of this bridge. Actually, this design office functions largely as an organization of specialists, and few designs of important structures are made that are entirely the work of one particular man or his immediate assistants. So it was with the Grand Coulee Dam spillway bridge design. The author had charge of the preliminary design of the bridge as a whole and the final design and stress analysis of the supporting arches; Senior Engineer John J. Hammond, in charge of the Concrete Dams Designing Section, supervised the design of the superstructure and the preparation of all construction drawings for the bridge; architectural designs of the bridge and epsecially the guard railings were made by the Structural Design Section of the Electrical and Mechanical Division under the direction of Senior Engineer Samuel Judd. The Materials, Testing, and Control Section, under the direction of Senior Engineer R. F. Blanks, supplied the necessary concrete technical data for the design of the structure. Gordon B. Kaufman of Los Angeles, California, was the consulting architect. The design was made under the general direction of K. B. Kenner, senior engineer on dams and J. L. Savage, chief designing engineer. All design and construction work of the Bureau is under the general direction of S. O. Harper, chief engineer, at Denver, Colorado, and John C. Page, commissioner, with headquarters at Washington, D.C.

*ACI JOURNAL, Jan. 1943; *Proceedings* V. 39, p. 149.

By HOMER M. HADLEY†

The writer has read Mr. Dexter's paper with a great deal of interest. It constitutes a very complete report on the design and construction methods employed in building what is in reality an unusually large bridge but one whose magnitude is dwarfed by the huge bulk of its foundation. It is a structure inherently strong and sturdy, both as a series of arches abutting against massive piers and in the rigidity and bracing wherewith the solid transverse cross walls connect the solid horizontal plate of the roadway with the solid barrels of the arches. In the position which it occupies, exposed at all temperatures to the moisture and spray from falling waters, this design which develops flat unbroken surfaces to a maximum and reduces exterior angles and corners to a minimum affords the greatest resistance possible to all those destructive agencies which we group under the name "weather." Built of the quality of concrete indicated by its test cylinders and to the thorough and conservative design which this paper outlines, it cannot but prove to be the permanent and enduring structure which it was the purpose of its builders to create.

Considering the massive monolithic character of its base it is difficult to conceive of differential earthquake movements being imparted to the arch segments, yet against this possibility there is a detail which the writer believes might be improved upon. This is the location and positioning of the $\frac{3}{8}$-in. round tie bars by which the layers of reinforcement on the upper and lower faces of the arch barrel are connected. Fig. 3 and 4 show the joints normal to the arch axis and the text makes no mention of shear keys having been provided. It is suggested therefore that another time ties be placed closely adjacent to the joints in each segment. If slight vertical movement in the plane of these joints were to occur without such nearby ties, the tendency would be to tear loose the mat of longitudinal bars on one side of the joint at the top surface and on the opposite side of the joint at the bottom surface. It is true that it is a highly remote contingency which this would guard against but it would be a simple precaution to adopt and with only a slight increase in the spacing of intermediate ties could be effected without use of additional labor or material.

At Santa Barbara in 1925 the writer saw the floor of a concrete building in which this very trouble developed. It was a floor of "pan" or ribbed type and a construction joint had been made in the middle of a first floor span above a large basement room. The joint ran vertically through and across each rib and being located at mid-span conformed fully with the requirement of standard specifications that construction joints be made at points of zero shear. Normally this was a point of zero shear but in the

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blind confusion of prodigious forces that is an earthquake it ceased to be such and nearly an inch of vertical slippage on the joint somehow occurred. Viewed from beneath one saw on one side of the joint the longitudinal bars of every joint bent downward and their encasing concrete torn back for a foot or two. A little stirrup close to the joint enclosing the bars and tying them up to the slab and the top of the rib would have prevented this. There would have been similar trouble topside if there had been any longitudinal bars there crossing the joint. At all events the writer thenceforth has been of the opinion that stirrups or ties closely adjoining construction joints are very desirable wherever there is a possibility of earthquake occurrence: i.e. practically everywhere.

It is noted in the paragraph on Arch Reinforcement that the spacing of ties used was not an arbitrarily chosen unit but on the contrary was the result of adherence to "Suggested Specifications for Concrete Arch Design." It may therefore seem impossible to employ the wider spacing of intermediate ties that the writer proposes. However against this view two considerations may be advanced: (1) that the committee which drafted the "Suggested Specifications" itself rather arbitrarily selected 30 main bar diameters as the maximum spacing, not being divinely inspired but being compelled as we humans are to exercise our best judgments and to make decisions; (2) that the heavy cover of concrete provided over the reinforcement on what correspond to slab conditions is greater than the Suggested Specifications would normally contemplate and therefore justifies a wider spacing of ties.

The handrails shown in Fig. 5 are of a novel, interesting type which the writer will look forward to seeing when installed. They ingeniously provide lighting for the bridge, of the modern tubular type, by locating it within the section of the top rails. Conventional lamp standards thereby being rendered unnecessary, conventional lamp standards are quite logically eliminated but it must not be overlooked that they, like many another useless thing in architectural design, might have been provided anyway regardless of lack of function simply because they were once and formerly provided. The small size and positioning of the lower rail bars indicates that thought was given to making them as inconspicuous as possible and for that the writer would express his sincere appreciation. Handrails on bridges constitute a real problem. Everyone undoubtedly has found himself riding in a closed trough or riding with an irritating series of obstructions and baffles to his vision when in crossing a bridge he could and should be enjoying commanding views. Everyone who swears has then heartily damned whoever he may be who is responsible for such stupidities. If nothing else is to be done there appear to be yet unexplored possibilities of obtaining necessary safety of traffic by means of curbs at

the sides of the roadway sufficiently high and strong to stop vehicles and then reducing railing sections to a size sufficient to provide safety for pedestrians and likewise to provide visibility. Where there are no sidewalks a combination of heavy base and light open top railing can be made. Should these proposals violate standard specifications then the writer has the temerity to say unhesitatingly, "Change the specifications." Frequently one encounters the attitude that specifications, standard specifications, are half sacrosanct. The breath is bated, tones of awe and reverence creep into the voice at their mention. Yet after all they are things of human creation and may be subject to improvement. Certainly the epitome of ultimate, final wisdom concerning bridge handrails has not yet found expression in specifications. Pending that outcome we may well continue to exercise our critical faculties.

The joint details shown in Fig. 7 to 9 impress the writer as being unusually neat and effective for the purpose their designer had in mind.

A closing word may be said about the equation of the arch rib, $Y = 0.003{,}253{,}3x^2 + 0.0771{,}242{,}09x^4$. Exactitude to a ten millionth part of x squared combined with a hundred trillionth part of x to the fourth power is refinement almost ethereal in its exquisite delicacy. Great was the Diana of the Ephesians but no less great is the goddess of our modern engineering, Mathematical Precision. Then to think that such ordinates must be used in combination with concrete strengths that increase with age and a changing modulus of elasticity value assumed only to the closest million or half million pounds per square inch! Sic transit gloria mundi!

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Some Factors Influencing the Strength of Concrete Containing Admixtures of Powdered Aluminum*

By CARL A. MENZEL†
Member American Concrete Institute

SYNOPSIS

This paper discusses the effects of small additions of aluminum powder on the strength of several types of concrete and concrete specimens. These effects result primarily from the generation of minute bubbles of hydrogen gas within the paste. This gas tends to expand the volume of the paste and increase the pressure within the mass which may be partially restrained by the confining parts of the mold. This tendency to expand the volume of the paste improves the intimacy of contact with adjacent aggregate particles and with embedded steel reinforcing bars. Under ordinary conditions the compressive strength of plastic concrete is sharply reduced with increase in porosity of the paste that occurs with unrestrained expansion. Under special conditions, particularly where the expansion is restrained, the strength may be increased by the admixture of aluminum powder. In a very special case of grouted gravel concrete, marked increases in compressive strength were obtained when the expansion of the grout was restrained by the interlocked pieces of gravel and the homogeneity of the mass for this type of concrete was greatly improved. With very lean mixes, or with mixes deficient in fine aggregate particles, slight improvements in strength were obtained with 6 by 12-in. cylinder specimens but these were small compared to the strengths obtainable with properly designed workable mixes of the same cement content but without aluminum powder.

INTRODUCTION

In an extensive investigation of factors influencing the results of pull-out bond tests it was found that the bond resistance developed by horizontal reinforcing bars held rigidly in pull-out specimens decreased rapidly as the depth of concrete under the bar increased. This decrease in bond resistance was found to be mainly due to the difference in the amount of normal settlement of plastic concrete under the bars—a phenomenon which results in large measure in water gain or "bleeding."

*Received by the Institute Dec. 9, 1942.
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Various means were tried to reduce or counteract this normal settlement of plastic concrete. One of the most effective was to add a very small amount of powdered aluminum to the concrete to produce the slight swelling in volume necessary to overcome the normal settlement under the fixed bars that occurs during sedimentation and compaction of the mix. This gradual swelling of the fresh concrete during the normal settlement period is the result of a well-known chemical reaction between small particles of aluminum and moist cement paste. It is the same type of reaction which has been used with much larger quantities of aluminum powder in plastic cement-sand mixtures to produce the lightweight concrete known as "Aerocrete."

The useful feature of this reaction is that with very *minute* quantities of aluminum powder it gradually produces small bubbles of hydrogen gas which remain essentially trapped within the paste.* The generation of these minute gas bubbles tends to increase the volume and the pressure of the confined paste thereby tending to improve the intimacy of contact of the paste with adjacent aggregate particles as well as with the embedded steel and the confining parts of the mold. Hence admixtures of aluminum powder can be expected to affect in various degrees the homogeneity and the strength of hardened concrete as well as the bond resistance of embedded steel bars with the enveloping concrete.

The ultimate extent and effect of the gas-producing reaction within the paste is subject to considerable manipulation. It is influenced by the amount and fineness of the aluminum powder, the type and fineness of the cement, the character of the concrete, and a variety of factors (internal and external to the concrete) acting to restrain the normal expansion of the paste between the aggregate particles. It is the purpose of this paper to present the results of tests which illustrate the conditions under which the strength of concrete is decreased or increased by an admixture of aluminum powder. A clear understanding of this matter is important for the intelligent use of this admixture because the substantial improvements in compressive strength that have been claimed for plastic concrete (40 to 60 per cent) have not been obtained even under exceptionally favorable and special conditions with standard 6 by 12-in. compression cylinders. It appears very improbable that such a great improvement in compressive strength will be realized in either pavement and in mass concrete, or in reinforced concrete. In fact the evidence indicates that the compressive strength of good plastic concrete will be lowered appreciably (about 15 to 20 per cent) by the smallest amount (about 2.0 grams per 100 lb. cement) of aluminum powder

*Many measurements of the change in level of the concrete at the top of 6 by 12-in. cylinders indicate that the aluminum powder reacts with the cement paste for about 1½ hours after mixing. The evolution of gas bubbles does not appear to influence the consistency or workability of the batch and hence the water required is not affected by the presence of the small quantities of aluminum required for improving bond resistance.

required to counteract settlement and improve the bond resistance of the upper bars in deep beams. More aluminum powder than this optimum amount causes a further sharp decrease in strength and will also reduce the bond resistance. It is very important therefore to limit the quantity of aluminum to the optimum required for good bond resistance.

GENERAL DESCRIPTION OF TESTS

Most of the tests reported herein were on standard 6 by 12-in. compression cylinders made for the most part in accordance with ASTM Specification C31-39. Variations from the usual procedure were introduced in some groups of tests to determine the effect of such factors as water leakage, restrained expansion, and pressure during hardening on the strength of concrete made with and without aluminum powder. Throughout these studies the test procedure followed was determined by the fact that the paste in concrete containing aluminum powder expands and that this expansion is subject to various degrees of restraint. Thus the studies included one group of tests in which 3 by 6, 4 by 8, 8 by 16, as well as the standard 6 by 12-in. cylinder specimens were used to determine the influence of variation in restraint introduced merely by altering the size of the compression test specimen. In another group the specimens were 6 by 36-in. cylindrical columns which later were sawed into four 6 by 9-in. compression cylinders to establish the effect of the restraint imposed by the weight of the concrete itself. Other groups of tests were designed to bring out the effect of different degrees of artificially imposed restraint and pressure.

The type of concrete employed ranged from a plastic workable machine-mixed concrete to the special grouted gravel concrete made with a cement-sand grout poured between ¾ to 1½-in. gravel pebbles. The grounted gravel was used in 6 by 12-in. cylinders and in 7 by 7 by 38-in. beams. Elgin sand and gravel aggregate was used for all types of concrete.

The majority of the cylinders were made with concrete of plastic consistency placed by hand rodding and having a cement content of 5 sacks per cu. yd., a slump of 5 to 6 in. and a W/C ratio of 0.63 by weight gross (net W/C about 0.54). The mixed aggregate was graded from 0 to 1-in. and consisted of 40 per cent sand graded 0 to No. 4 sieve.

In one group of cylinders the concrete was made richer in cement content (6.5 sacks per cu. yd.) and of a stiffer consistency with a slump of about 2.8 in. In this group the mixed aggregate was graded 0 to 1½ in. and consisted of 37 per cent of sand graded 0 to No. 4 sieve. In another group of cylinders the cement content ranged from 3 to 5.5 sacks per cu. yd. but the concrete was purposely made deficient in sand which produced harsh mixes although the slump was practically constant at 5 to 5.5 in. In this group the mixed aggregate was graded 0 to 1 in.

with sand percentages too low (30 to 40 per cent) in any given mix for the amount of cement present in the mix.

The following paragraphs give further information on the materials and the procedures followed in determining the effect of the various factors studied in connection with the use of aluminum powder. Further details on some items are given in notes accompanying the individual diagrams.

Cement

All specimens were made with the laboratory mixture consisting of equal parts by weight of 4 commercial brands of normal strength portland cement (ASTM Type 1) purchased in Chicago. The surface area of the mixture was approximately 1700 sq. cm. per gram and the alkali content totaled .86 per cent (Na_2O — .29 per cent; K_2O — .57 per cent). The different groups of tests were made at various times over a period of 3 years. Because of this it happened that the same lot of cement was not used in all of the tests, but the changes in the physical and chemical properties of this blend of the same 4 cements are very small from year to year.

Curing

All specimens were cured moist and tested damp at age of 7 days except in one group which included 28-day tests. The temperature of the freshly mixed concrete and that of the moist room was maintained at 70 to 72 F.

Aggregate

The Elgin sand and gravel used is a typical calcareous aggregate with particles ranging from regular to irregular in shape but generally having well-rounded edges. Surfaces varied from a smooth, slippery texture to a rough, porous, pitted type with neither type predominating. Both the sand and gravel were oven dried, screened to selected sieve sizes and recombined to maintain the constant grading desired. The moisture content of the prepared sizes available in the storage bins was at or very near the oven-dry condition. For this reason the W/C ratios given are based on the gross water used in the mix. For the plastic concrete used in most of the studies the gross W/C value of 0.63 would be reduced to about 0.54 if allowance is made for absorption of water by the dry aggregate used.

Aluminum Powder

Whenever aluminum powder was used the desired amount (in grams per 100 lb. cement) was premixed by hand with the separately controlled finest sand portion (0-No. 100 sieve) of the batch. This mixture was then added directly to the dry aggregate and cement already in the mixer. In all except one group of tests, the aluminum powder was of the

polished type. The characteristics of the four types used are given at the top of Fig. 2.

Mixing

All machine mixed concrete was mixed dry ½ min. and wet for 2 min. in a 1½-cu. ft. Lancirick open tub-type mixer. The concrete batch was allowed to stand an additional 2 or 3 min. while the slump and remolding effort tests were being made. Usually the concrete was placed in the first mold within 6 minutes after the beginning of mixing.

Cylinder Molds

These were made from cold drawn seamless steel tubing in accordance with ASTM Specification C31-39. The smooth interior surfaces were coated with a thin layer of vaseline to prevent adhesion with the concrete. Usually all joints were carefully sealed against leakage of water or paste with a rosin-paraffin mixture. In some special tests the molds were purposely left unsealed to determine the effect of leakage.

Rodding

All cylinders of machine-mixed concrete, whether 3 by 6-in., 4 by 8-in., 8 by 16-in., or the standard 6 by 12-in. size were rodded in 3 layers. The special 6 by 30-in. cylindrical columns were rodded in 5 layers.

Capping

Immediately after completion of rodding the concrete was struck off flush with the top of the mold. Thereafter it remained undisturbed and the leveled surface was free to fall or rise except for the adhesive and frictional resistances developed by the wet concrete in contact with the lubricated, non-absorptive smooth surface on the interior of the steel cylinder mold. The cylinders were carefully covered with a blanket of damp burlap to prevent moisture loss from the fresh concrete by evaporation during the early hardening period. The wet burlap in turn was covered with a dry burlap to minimize moisture loss and temperature changes due to the wet-bulb effect that would be introduced by free evaporation from the uncovered damp burlap. After 20 hours in molds the cylinders were removed to the moist room where the top ends were capped in the standard manner. This delay in capping was necessary to eliminate the possibility of direct restraint which a heavy capping plate would introduce against the expansion of concrete containing aluminum powder if capped shortly after placing as is often done. However, in some special tests, the molds were purposely covered with one or more plates immediately after the concrete was struck off in order to determine the effect of restraints of various magnitudes.

DISCUSSION OF RESULTS

To conserve space and the time of the reader, the test data are presented graphically in Fig. 1 to 9 inclusive, the salient features of each of which will now be discussed.

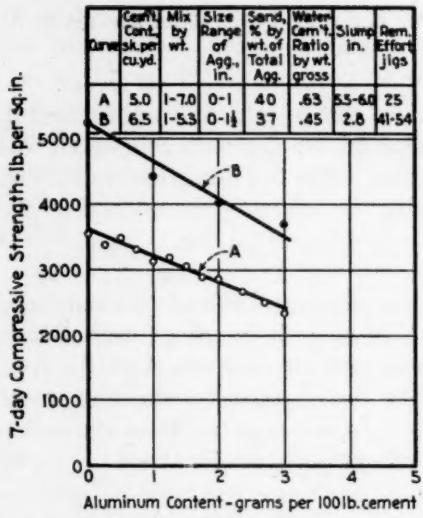

Fig. 1—Effect of powdered aluminum on compressive strength of plastic concrete

Aluminum Powder: Polished type. Specimens: 6 by 12-in. concrete cylinders placed by hand rodding in sealed molds. Cured moist and tested damp at age of 7 days. Aggregate: Elgin sand and gravel.

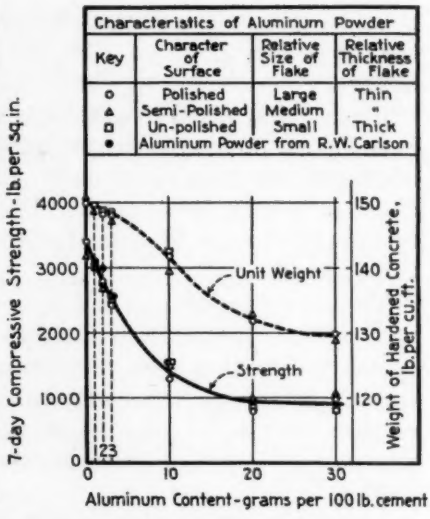

Fig. 2—Relation between aluminum content and the compressive strength and unit weight of plastic concrete

Aluminum Powder: Four different types as indicated. Specimens: 6 by 12-in. concrete cylinders placed by hand rodding in sealed molds. Cured moist and tested damp at age of 7 days. Concrete: 5 sacks cement per cu. yd., 5-6-in. slump (W/C = 0.63 by weight); sand and gravel aggregate graded 0-1-in.

Effect of powdered aluminum on compressive strength of plastic concrete.

Both curves A and B of Fig. 1 show a steadily decreasing compressive strength with increasing additions of minute quantities of aluminum powder to plastic concrete. These curves are based on 7-day strengths of standard 6 by 12-in. concrete compression cylinders cured moist and tested damp. Precautions were taken in the making of these cylinders to permit the concrete containing aluminum powder to expand vertically as freely as possible without external restraint other than that imposed by the mold itself and the internal resistance of the mass. Thus under the conditions of these tests the expansion of the concrete within the mold was restrained (1) laterally by the walls of the mold, (2) vertically by the adhesional and frictional resistances developed by the contact of the concrete with the walls of the mold and, (3) by the internal resistances to movements of particles within the mass. It will be seen from curve A that the compressive strength of the 5-6 in. slump concrete containing 5 sacks cement per cu. yd. decreased from about 3600 psi for no aluminum powder to about 2400 lb. with the addition of only 3 grams of aluminum per 100 lb. of cement—a reduction in strength of about 33 per cent. Curve B for a somewhat richer and stiffer concrete, simulating paving concrete, showed approximately a 30 per cent reduction in com-

pressive strength with the addition of 3 grams aluminum per 100 lb. of cement. It appears from these curves that the 7-day compressive strength of good concrete of plastic consistency varying in cement content from 5.0 to 6.5 sacks per cu. yd. was reduced roughly 10 per cent for each gram of aluminum powder added per 100 lb. of cement. The aluminum powder used in these tests was of the polished type with flat flakes commonly sold by paint stores as "aluminum bronze."

Relation between aluminum content and the compressive strength and unit weight of plastic concrete.

These relations are shown by the curves of Fig. 2 for aluminum additions up to 30 grams per 100 lb. of cement and for four different types of aluminum powder varying in character of surface, and in size and thickness of the flakes. Notwithstanding the substantial variations in the characteristics of the 4 types of aluminum powder it is apparent that the plotted points for each type may for all practical purposes be represented by single curves for strength and unit weight as shown in the diagram.

Although all mixes represented in Fig. 2 initially had a cement content of 5.0 sacks per cu. yd. the subsequent expansion of the concrete containing aluminum powder lowered the cement content of the hardened concrete in direct proportion to the reduction in unit weight. This reduction in unit weight and cement content was about 2 per cent for 3 grams of aluminum powder and about 6, 12 and 14 per cent respectively for 10, 20 and 30 grams of aluminum powder per 100 lb. of cement. The cement content of the hardened concrete was lowered about one-tenth of a sack per cu. yd. for 3 grams of aluminum and 0.3, 0.6, and 0.7 sacks per cu. yd. respectively, for 10, 20 and 30 grams of aluminum. The composite strength curve of Fig. 2 for the 4 types of aluminum powder indicates an average reduction in compressive strength of about 26 per cent with the addition of 3 grams of aluminum or about 9 per cent per gram. This progressive strength reduction is believed to be the direct result of increase in porosity of the paste caused by gas evolution and not to the accompanying reduction in cement content of 2 per cent or less of the hardened concrete.

Effect of imposed restraint to expansion on compressive strength of plastic concrete containing powdered aluminum.

The effect of restraint imposed at the top of the mold is revealed by comparing the curves in Fig. 3 for potential restraints of .30 and 2.0 psi. with the curve for "no restraint." The latter curve shows the same progressive reduction in compressive strength with aluminum additions as the curves of Fig. 1 and 2. However, when the tops of the freshly molded cylinders were covered by one or more steel plates, the downward trend in strength was reversed when the expansion of the concrete was sufficient to bear against the plates. The compressive strength then

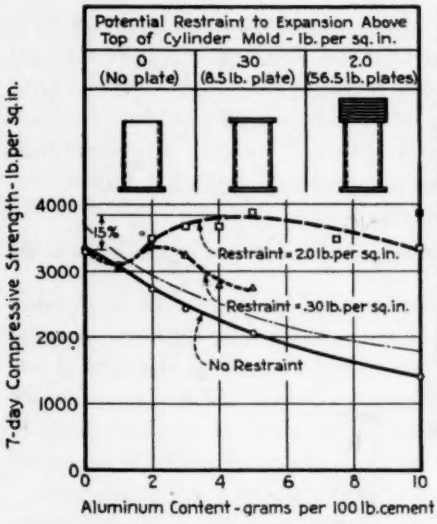

Fig. 3—Effect of imposed restraint to expansion on compressive strength of plastic concrete containing powdered aluminum

Aluminum Powder: Polished type. Specimens: 6 by 12-in. concrete cylinders placed by hand rodding in sealed molds except for specimens represented by light dot-and-dash line which were made with unsealed molds. Concrete: 5 sacks cement per cu. yd.; 5-6-in. slump (W/C = 0.63 by weight); sand and gravel graded 0-1-in. Imposition of Restraint: Immediately after completion of rodding the concrete was struck off flush with the top of the mold in the usual manner. One or more capping plates were then placed on the top edge of the mold as illustrated, to provide a potential restraint of 0.30 and 2.0 psi. to expansion of the concrete containing aluminum powder. With some aluminum contents the expansion was fully restrained but with others it was sufficient to lift the superimposed plates above the top edge of the mold until equilibrium between the expansive and restraining forces was attained. After 20 hours the cylinders were removed from the molds and stored in the moist room where the top ends were capped in the usual manner for test at the age of 7 days. Solid square plotted value represents compressive strength obtained with potential restraint of .41psi imposed on top surface of concrete.

increased with aluminum content until the force of the expansion was sufficient to lift the cover plates off the top edges of the mold. For further increase in the aluminum content the curves show a decrease in compressive strength because the cover plates were lifted higher and the concrete expanded a greater amount before equilibrium between the expansive and the external restraining forces was attained. It would appear that if a greater restraint than that employed (2.0 psi.) had been imposed, the compressive strength would not have fallen off as shown in Fig. 3.

In this connection it is interesting to record that for plastic concrete of the type represented in Fig. 3 (as well as in most of the tests in other groups reported herein) the normal settlement for no aluminum powder was about $\frac{1}{16}$ in. below the top of the sealed 6 by 12-in. cylinder mold. With 1 gram of aluminum the expansion of the concrete was only enough to overcome the normal settlement. For higher aluminum contents the unrestrained expansion above the top of the mold was about as follows: $\frac{3}{32}$ in. for 2 grams, $\frac{3}{16}$ in. for 3 grams, and $\frac{5}{8}$ in. for 10 grams. Compared with cylinders made without aluminum powder the final expansion in the hardened condition was about $\frac{1}{16}$ in. for 1 gram and $\frac{5}{32}$, $\frac{1}{4}$ and $\frac{11}{16}$ in. respectively for 2, 3 and 10 grams of aluminum per 100 lb. of cement.

It is also of interest that with 10 grams of aluminum the cover plates providing a restraint of 2.0 psi. were lifted from $\frac{1}{8}$ to $\frac{3}{16}$ in. above the top edges of the mold but that in comparable tests with twice as many plates (restraint 4.0 psi.) the plates were not lifted. With this higher restraint the 7-day compressive strength was about 3850 psi. as indicated by the single solid square in Fig. 3.

Although the effect of restraining the expansion of plastic concrete containing alumnium powder is such that the compressive strength of concrete with 5 grams of aluminum was nearly doubled by imposing a restraint of 2.0 psi. (increase from 2000 to 3900 psi.) it is important to point out that this value is only 15 per cent higher than the compressive strength of corresponding concrete made without aluminum powder and not subjected to an imposed restraint. It will be shown by the results discussed later and presented in Fig. 4 that the compressive strength of plastic concrete made without aluminum powder can be increased over 25 per cent by the direct application of an external pressure of 2.0 psi. Hence, if improvement in strength is the main objective, it appears that superior results will be obtained under comparable conditions by manipulations involving the direct application of pressure to plain concrete without relying upon aluminum powder to create internal pressure.

Attention is directed to the light dot and dash line curve in Fig. 3 approximately parallel to the solid line curve for tests with "no restraint." The latter represents tests with molds having all joints sealed to prevent leakage of water. In contrast the dot and dash-line curve represents similar tests made with molds in which all joints were left unsealed to permit leakage of water. It will be seen that for all mixes the compressive strength was higher with leakage of water than without. Corresponding tests with restraint and with water leakage are only available for a few of the mixes and hence are not included in Fig. 3 for comparison with the relations shown by the heavy dotted and heavy dash-line curves. It may be said, however, that in all cases higher compressive strengths were obtained when water leakage occurred than without water leakage as in the tests made without restraint.

Effect of external pressure on compressive strength of plastic concrete without aluminum powder.

The results in Fig. 3 showed that when the expansion of plastic concrete containing aluminum powder is purposely restrained the concrete is subject to internal pressure during the early hardening period which tends to increase its compressive strength to values higher than that of similar concrete without aluminum powder. The curves of Fig. 4 are of interest in showing that the compressive strength of plastic concrete made without aluminum powder may be increased substantially by applying external pressure as described in the notes accompanying Fig. 4.

The two curves in the left diagram of Fig. 4 show that the strength of concrete made in unsealed molds which permitted leakage of water to occur was about 10 per cent higher than when all joints of the mold were sealed to prevent leakage. However, both curves show increased strength with imposed pressure during hardening. With pressures of 2.0 and 4.0 psi. the strengths were increased 21 and 26 per cent respectively for

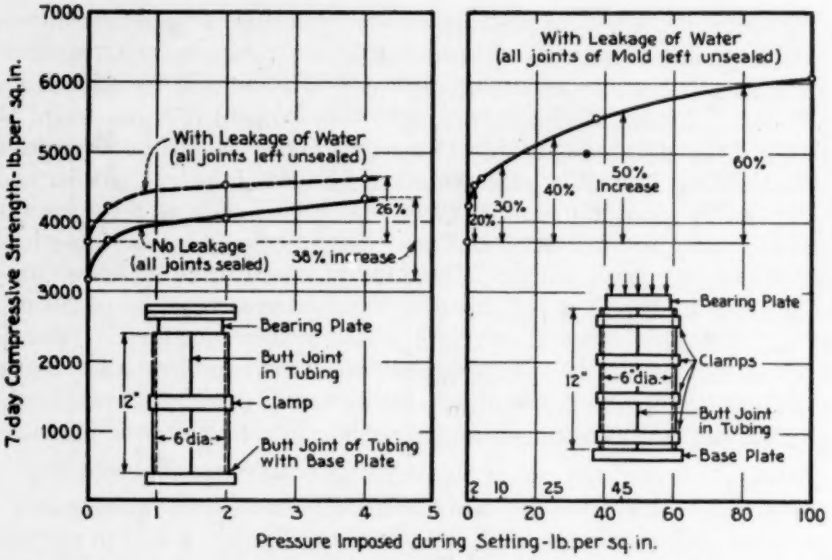

Fig. 4—Effect of external pressure on compressive strength of plastic concrete without aluminum powder

Aluminum Powder: None. Specimens: 6 by 12-in. concrete cylinders cured moist and tested damp at age of 7 days. Concrete: 5 sacks cement per cu. yd., 5-6-in. slump (W/C = 0.63 by weight) sand and gravel aggregate graded 0-1-in.

Left Diagram: Test cylinders represented in the left diagram were made with both sealed and unsealed molds. Immediately after completion of rodding the concrete was struck off flush with the top of the mold. Pressures of 0.30, 2 and 3 psi. were then applied and maintained for 20 hours by placing a steel plate 1 ¼-in. thick and 5¾-in. in diameter on top of the fresh concrete. The diameter of this plate was purposely made ¼-in. smaller than the internal diameter of the mold to minimize binding as the concrete settled under continued pressure. The magnitude of the pressure was controlled as desired by using additional plates. After 20 hours the plates were removed, the cylinders placed in moist room, and the top ends capped in the usual manner.

Right Diagram: In the right diagram the plotted points represent cylinders made in unsealed molds. The four points on the curve for pressures of 0, 0.30, 2, and 4 psi. are the same as those on the dash-line curve in the left diagram. The two remaining points represent tests in which an average initial pressure of 38 and 100 psi. respectively was applied in a testing machine. Pressure was applied simultaneously to the three companion cylinders for each condition by placing the cylinders one above the other but separated by a steel bearing plate as shown in diagram. Each set of three cylinders remained under pressure for 20 hours during which period the pressure dropped to 31 and 95 psi. respectively from the initial values of 38 and 100 psi. Solid circle represents compressive strength of cylinders subjected to 35 psi pressure for only 5 minutes.

unsealed molds. For sealed molds the strengths were increased 28 and 38 per cent respectively by the same pressures.

The curve in the right diagram of Fig. 4 shows the effect for unsealed molds of using higher external pressures than those shown in the left diagram of this figure. This curve is based on tests with unsealed molds because it would be very difficult to avoid leakage of water from ordinary molds at the high pressures of 38 and 100 psi. employed in these tests. It is seen that for pressures of approximately 10, 25, 45 and 80 psi. the strength of plastic concrete was increased 30, 40, 50 and 60 per cent respectively. In actual concreting operations pressures of 10 psi. might be attained under favorable conditions at the bottom of 10-ft. lifts of fresh concrete.

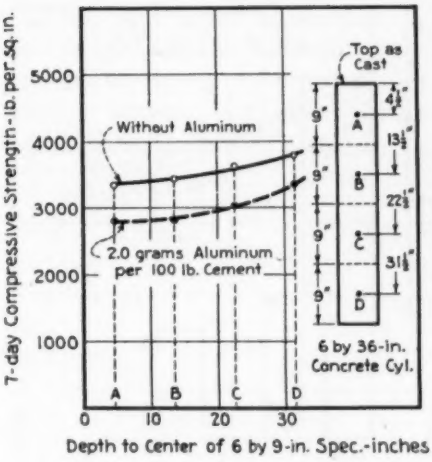

Fig. 5—Effect of weight of super-imposed concrete on compressive strength of specimens 9 in. high sawed consecutively from a 6 by 36-in. concrete cylinder

Aluminum Powder: Polished type; Concrete: 5 sacks cement per cu. yd., 5-6-in. slump (W/C = 0.63 by weight) sand and gravel graded 0-1-in. Specimens: The 6 by 36-in. concrete cylinders were made in sealed steel molds of the same type as for standard 6 by 12-in. compression cylinders. Each cylinder was rodded in 3 layers. Cylinders cured in the mold for 20 hours and then in moist room until 5 days old when they were sawed into four cylinders 9 in. high as shown in diagram. Sawed specimens were immediately returned to the moist room where the sawed ends were capped for compression tests at 7 days. Each plotted point is based on three 6 by 9-in. specimens tested several weeks apart.

As indicated by the notes accompanying Fig. 4 the cylinders remained under pressure for 20 hours. In the right diagram of Fig. 4 a solid circle is plotted which represents the average strength of 3 cylinders which were subjected to a pressure of 35 psi. for only 5 minutes. With this substantially shorter period of pressure application the strength was increased from about 3750 to 5000 psi. or about 33 per cent. The nearest point on the curve plotted through the open circles indicates that the application of pressure for 5 minutes developed approximately 92 per cent of the strength of corresponding cylinders which remained under similar pressure for 20 hours. Time under pressure therefore, does not appear to be an important factor. This appears logical because most of the water is squeezed from the concrete during the first few minutes of pressure application.*

Effect of weight of superimposed concrete on compressive strength of plastic concrete.

The two curves of Fig. 5 for concrete made with and without aluminum powder each show a gradual increase in the compressive strength of specimens 9-in. high, sawed consecutively from the top end of 6 by 36-in. concrete cylinders. It will be clear from the diagram showing the locations of the sawed specimens in the original cylinder that each successive specimen was subjected to the weight of the plastic concrete above it. In plotting the results the depth of the superimposed concrete above the center of each specimen was used for abscissa since this is proportional to the average pressure to which each specimen was subjected during setting.

Although these results provide further evidence that the strength of concrete both with and without aluminum powder is increased by pres-

*The general effects of application of pressure to fresh concrete were reported by D. A. Abrams in the following publications: Bulletin 71, Engineering Experiment Station, University of Illinois and Bulletin 3, Structural Materials Research Laboratory, Lewis Institute, Chicago.

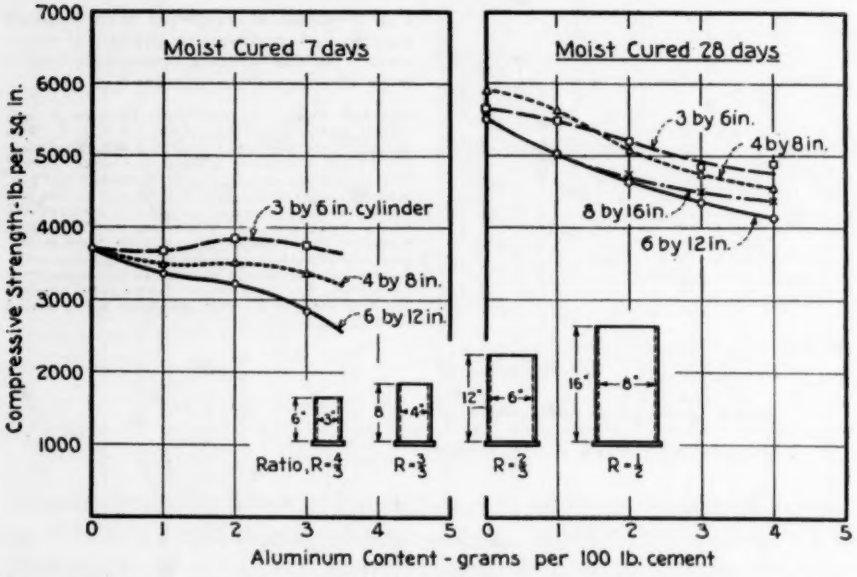

Fig. 6—Effect of size of test cylinder on compressive strength of plastic concrete containing powdered aluminum

Aluminum Powder: Polished type; Concrete: 5 sacks cement per cu. yd.; 5-6-in. slump (W/C = 0.63 by weight); sand and gravel graded 0-1-in.

Specimens: For each aluminum content four concrete cylinders of each size were made simultaneously from the same batch and with equal care. For each size of cylinder the concrete was rodded in 3 layers in sealed steel molds which were of the same type as for the standard 6 by 12-in. cylinder. After striking off flush with the top of the mold the concrete was free to fall or rise except for the adhesive and frictional resistances developed by the wet concrete in contact with the lubricated, non-absorptive, smooth steel surface of the interior of the mold.

In general, for cylinders of diameter d, and height 2d, the ratio, R, of the area of the vertical surface of the interior of the cylinder mold to the volume of the mold will be equal to 4/d. Hence, for 3 by 6-in., 4 by 8-in., 6 by 12-in., and 8 by 16-in. cylinders R = 4/3, 3/3, 2/3 and 1/2 respectively as shown in the diagram. On this basis the 3 by 6-in. mold provides 2 times and the 4 by 8-in. mold 1.5 times as much surface per unit volume of concrete as the standard 6 by 12-in. mold, while the 8 by 16-in. mold provides about ¾ as much surface as the 6 by 12-in. mold. The potential adhesive and frictional resistance to vertical movement of the concrete that results from expansion with aluminum powder can therefore be expected to be relatively greater with small cylinders than with standard 6 by 12-in. cylinders, or with the still larger 8 by 16-in. cylinders. All specimens were cured moist and tested damp at age of 7 days or 28 days.

sure the extent of the increase is lower than might be expected from the curve in the left diagram of Fig. 4 for "no leakage." As indicated in the discussion of that diagram the curve showed an increase in compressive strength of about 28 per cent for an external pressure of 2.0 psi., whereas specimen D in Fig. 5 which hardened under a somewhat higher potential pressure (2.3 psi. at base of 27-in. high column of concrete) showed an increase of only 14 per cent. Specimen D for aluminum powder shows an increase in compressive strength of about 19 or 20 per cent. It appears quite possible that only a portion of the superimposed weight of concrete is effective in a long cylinder specimen because of frictional and adhesional resistance at the surface of the cylinder walls which impedes vertical movement of the concrete in either an upward or downward direction. In any event the curves of Fig. 5 provide direct experimental evidence

that with concrete containing aluminum powder the compressive strength of a column will be substantially lower at the top of the lift where the expansion is least restrained than at the bottom of the lift where both restraint and superimposed weight come into play.

Effect of size of test cylinder on compressive strength of plastic concrete containing powdered aluminum.

The curves in both diagrams of Fig. 6 are based on strictly comparable tests to determine the influence of size of cylinder mold on the relation between aluminum content and compressive strength of plastic concrete. The left diagram presents data on specimens moist-cured and tested damp at 7 days, the same curing and age of test used for the specimens represented in the other figures of this paper. The right diagram is based on specimens which were moist-cured and tested damp at the age of 28 days. Although 7 days of continuous moist curing is believed to represent the equivalent of good curing in many concreting operations, it was of interest to determine to what extent the clearly-established trends indicated by the 3 curves for the 7-day tests would be maintained by extending the period to 28 days.

A group of specimens for the 28-day tests was made several days after the results of the 7-day tests became available, and, in view of the trends indicated by the curves for the 7-day tests, was enlarged to provide an additional size of test cylinder (8 by 16-in.) and an additional aluminum content (4 grams per 100 lb. of cement). It should be pointed out that the strength value of each plotted point in both diagrams is based on the average strength of 4 specimens except for the values for the 28-day specimens made with 0 per cent aluminum which were based on 6 test cylinders of each size. Since each point was so well established and since the test data in the two diagrams are strictly comparable the differences in results indicated by the 7 and 28-day tests are attributed directly to changes in the structure of the porous paste that occurred with longer moist curing and not to variations incident to the making of the specimens. Because 7 days of moist curing is more representative of the usual operations than 28-days moist curing, the results of the 7-day tests are believed to be of more practical value.

The 3 curves in the left diagram of Fig. 6 indicate that the size of the cylinder mold is an important factor influencing the relation between aluminum content and compressive strength of plastic moist-cured concrete tested damp at age of 7 days. This difference in the indicated compressive strength of specimens from the same batch of concrete is believed to be due to a difference in the degree of restraint offered by the different sizes of cylinder mold. As explained in the notes accompanying Fig. 6, the 3 by 6-in. mold provides 2 times and the 4 by 8-in. mold 1.5 times as much vertical surface at the interior of the mold per unit of volume of

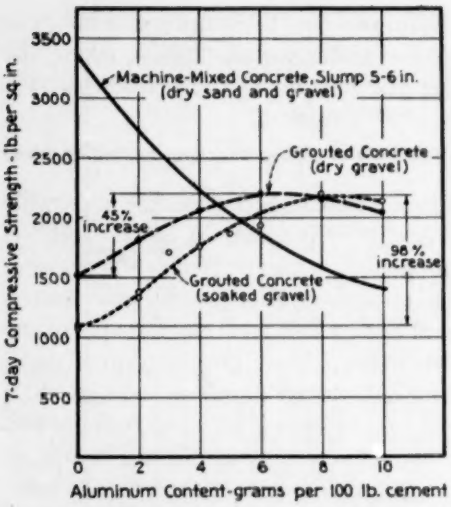
Machine-Mixed Concrete, Slump 5-6 in. (dry sand and gravel)
Grouted Concrete (dry gravel)
45% increase
98% increase
Grouted Concrete (soaked gravel)
7-day Compressive Strength - lb. per sq. in.
Aluminum Content - grams per 100 lb. cement

Fig. 7—Comparison of the effect of powdered aluminum on compressive strength of machine-mixed plastic concrete and grouted concrete

The curve for the machine-mixed plastic concrete is based on the tests of 6 by 12-in. cylinders represented by the open circles in Fig. 2 for polished aluminum powder. The cement content of this concrete was 5 sacks per cu. yd. for no aluminum and decreased progressively with increase in aluminum content to a value of approximately 4.7 sacks cement per cu. yd. for 10 grams aluminum per 100 lb. cement.

The curves for the grouted gravel concrete are based on tests of three 6 by 10½-in. cylinders for each plotted point. These specimens were made as 6 by 12-in. molds but the rough top 1½ in. of the grouted gravel was sawed off at the age of 5 days to provide a smoother and more uniform surface for capping. The cement content of this concrete was between 4.7 and 5.0 sacks per cu. yd. and the same polished aluminum powder was used as for the machine-mixed plastic concrete.

All cylinders were made in sealed molds and were moist cured and tested damp at the age of 7 days.

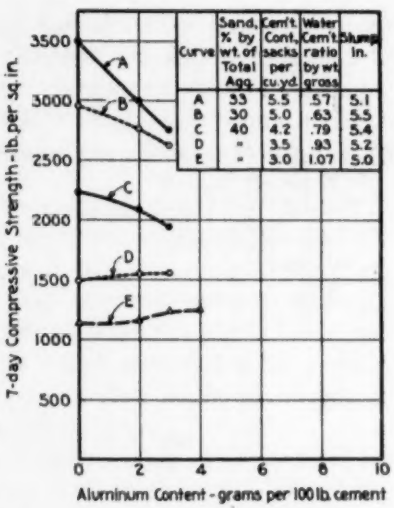
Curve | Sand, % by wt. of Total Agg. | Cem't. Cont., sacks per cu. yd. | Water Cem't ratio by wt. gross | Slump, in.
A | 33 | 5.5 | .57 | 5.1
B | 30 | 5.0 | .63 | 5.5
C | 40 | 4.2 | .79 | 5.4
D | - | 3.5 | .93 | 5.2
E | - | 3.0 | 1.07 | 5.0
7-day Compressive Strength - lb. per sq. in.
Aluminum Content - grams per 100 lb. cement

Fig. 8 (right, above)—Effect of aluminum powder on compressive strength of concretes of varying sand and cement contents

Aluminum Powder: Polished type; Specimens: 6 by 12-in. concrete cylinders placed by hand rodding in sealed molds. Cured moist and tested damp at age of 7 days. Aggregate: Elgin sand and gravel graded 0-No. 4 sieve and No. 4 sieve to 1-in. respectively. The percentage of sand and other details of the concrete for the different mixes and curves are tabulated in the diagram. Consistency: Although the slump was practically constant at 5 to 5½ in. the consistency of mixes B, C, D and E became progressively harsher because of intentional reduction in sand and cement contents. None of these four mixes could be considered to be good plastic concrete.

concrete as the 6 by 12-in. mold. Hence, it can be expected that the 3 by 6-in. mold offers to a greater degree the adhesional and frictional resistance which tends to restrain expansion than does the 4 by 8-in. or the 6 by 12-in. mold.

These results suggest that small test cylinders do not give reliable indications of the compressive strength of plastic concrete containing aluminum powder under the usual concreting operations, particularly in flat slabs involving a minimum of restraint from the sides of the mold.

The 4 curves in the right diagram of Fig. 6 for 28-day tests indicate, as in the case of the 7-day tests, somewhat higher strengths for the 3 by 6-in. and 4 by 8-in. cylinders than for the 6 by 12- and 8 by 16-in. cylinders but the differences between individual curves for each size of specimen are less. They do show, however that the compressive strength

of plastic concrete moist-cured for 28 days decreases progressively with increase in aluminum content whether the test cylinders are larger or smaller than the standard 6 by 12-in. cylinder generally used for compression tests.

Comparison of the effect of powdered aluminum on compressive strength of machine-mixed plastic concrete and grouted concrete.

The curves of Fig. 7 show the striking difference between the effect of aluminum powder on the compressive strength of two widely different types of concrete. For the machine-mixed plastic concrete the strength decreases progressively with each increment of aluminum powder whereas for the grouted concrete the strength increases progressively with aluminum content beginning with much lower values but ultimately attaining much higher values than for the plastic concrete of the same cement and aluminum content.

The reduction in strength of the plastic concrete is explained by the increase in porosity and decrease in the strength of the paste that occurs with unrestrained expansion. The low strength of the grouted gravel without aluminum powder is due partly to the high water content of the grout (W/C = .71, 8 gal. per sack) and partly to the fact that notwithstanding the high degree of fluidity of the grout it is not possible to fill completely the voids in the gravel. Furthermore, after the grout is placed the paste locked within each void tends to settle out of contact with the overlying gravel and to form small permanent cavities which leave the individual pieces of aggregate without the substantial support and bedding needed to tie the mass together effectively.

When aluminum powder is added to the grout, the latter tends to expand along the lines of least resistance into all hollow spaces and ultimately with enough aluminum powder the expansion is sufficient to exert a pressure against the adjacent gravel particles. The progressive improvement in the contact and embedment of the gravel in the matrix of expanded grout results in the progressive increase in compressive strength shown by the curves of Fig. 7 until optimum values are attained at about 6 grams of aluminum for the dry gravel and about 8 grams of aluminum for the soaked gravel. Further increase in aluminum content beyond these optimum values is believed to result in either an expansion of the whole mass or in the inability of the gravel to keep the paste locked within the void spaces.

It is interesting to note that the dry gravel developed a substantially higher initial strength without aluminum powder than the soaked gravel. This is believed to be partly due to a reduction in the water content of the paste that results from absorption of water by the dry gravel and

partly to a more intimate contact established between the grout and the surface of the dry gravel due to a suctional force of the same type as that which holds plaster to an absorptive base. With the soaked gravel the initial advantage of dry gravel is gradually reduced by the pressure of the expanding paste, and ultimately about the same strengths were attained at optimum values of aluminum content. Note that increase in strength of the grouted gravel concrete ranges from 45 to 98 per cent depending on whether the gravel was initially dry (oven dry) or soaked.

It should be pointed out that the cement content of the two types of concrete represented in Fig. 7 was very nearly constant varying from about 4.7 to 5.0 sacks per cu. yd. of hardened concrete of each type. The high strength of the grouted concrete with 8 or 10 grams of aluminum (about 2200 psi.) compared with about 1500 psi. for the machine-mixed plastic concrete appears to be mainly due therefore to the fact that the expansion of the paste in the grouted concrete was substantially restrained. On the other hand, it should be noted that the strength of the machine-mixed plastic concrete made without aluminum powder produced concrete about 50 per cent stronger than that obtained at optimum aluminum contents with the grouted gravel.

Effect of aluminum powder on compressive strength of concretes of varying sand and cement contents.

The different mixes A, B, C, D and E used in this group of tests will evident from the table in Fig. 8. As indicated by the notes accompanying this figure, the slump for the different mixes was practically constant at 5 to 5½ in. but the consistency of mixes B, C, D and E became progressively harsher because of the intentional reduction in sand and cement. Consequently none of these four mixes could be considered to be good plastic concrete.

The respective strength curves for these mixes indicate a progressive change in the effect of aluminum powder as the sand or cement content was reduced. Curve A shows the familiar reduction in compressive strength that was obtained with plastic concrete containing aluminum powder. With curves B and C the reduction is less pronounced. Curves D and E reveal trends toward an increase in strength with the aluminum additions represented. Curves D and E represent mixes which simulate to a slight degree the conditions present in the grouted concrete discussed in Fig. 7. Apparently there was not enough paste in these mixes to provide for proper embedment of the coarse aggregate particles. With aluminum additions the paste expanded sufficiently to overcome this initial deficiency in paste volume. However, further increments of aluminum would probably cause the concrete mass as a whole to expand and under these conditions the curves for mixes D and E might be

expected to follow a downward trend in compressive strength. It will be evident (from results of Fig. 6 for 7-day tests) that if the harsh mixes D and E had been molded into 4 by 8 or 3 by 6-in. cylinders the relative improvement in strength with aluminum additions would be substantially higher than is indicated by the curves of Fig. 8 for 6 by 12-in. cylinders. This remark is offered as a possible explanation of the greater improvements in compressive strength reported by some investigators with the admixture of aluminum powder.

It may be pointed out that with mixes having a greater deficiency in sand or cement content than mixes D and E the strength would be initially lower and the relative increase appreciably higher with the aluminum additions indicated in this figure. It will be clear, however, that such mixes would be even harsher than mixes D and E and entirely impractical for usual concreting operations requiring concrete having good workability. The use of aluminum powder to improve the strength of such harsh mixes would be unwarranted since much higher strengths could be obtained with the same cement content by proper adjustments in the percentage of sand in the mix.

Effect of aluminum powder on the modulus of rupture and compressive strength of grouted gravel concrete.

The two curves in the left diagram of Fig. 9 indicate some improvement in the modulus of rupture of grouted concrete by the use of aluminum powder in the grout. The greatest relative improvement in modulus of rupture is indicated by the curve for soaked gravel although the plotted points are somewhat scattered indicating that erratic results are possible even when the specimens are carefully made under closely comparable conditions in the laboratory. Relatively little improvement in modulus of rupture is indicated by the curve for dry (oven-dry) gravel and one plotted point for tests with 6 grams of aluminum powder per 100 lb. of cement falls well below the curve. Note that the modulus of rupture for dry gravel and no aluminum powder is about the same as that for the highest strength developed with 8 grams of aluminum powder in tests with soaked gravel. These results like those for grouted gravel in Fig. 7 indicate the advantage of using dry gravel in grouting operations.

The two graphs in the right diagram of Fig. 9 indicate a somewhat greater improvement with aluminum powder in the compressive strength of modified cubes when the gravel was soaked than when dry. However, the improvement is small compared with increases of 45 and 98 per cent in compressive strength indicated by the curves in Fig. 7 for grouted gravel tested in the form of 6 by 10½-in. cylinder specimens. It is well to point out that the modified cubes were tested with load applied to the sides of the beam as cast (Fig. 7) whereas the cylinders were loaded in the position as cast.

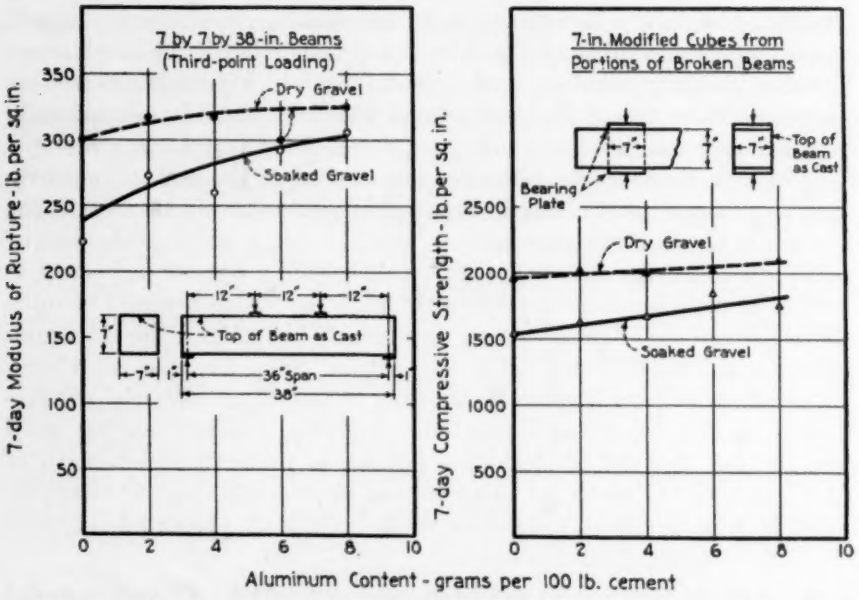
7 by 7 by 38-in. Beams
(Third-point Loading)
Dry Gravel
Soaked Gravel
7-day Modulus of Rupture - lb. per sq. in.
Top of Beam as Cast
36" Span
38"
7-in. Modified Cubes from
Portions of Broken Beams
Bearing Plate
Top of Beam as Cast
Dry Gravel
Soaked Gravel
7-day Compressive Strength - lb. per sq. in.
Aluminum Content - grams per 100 lb. cement

Fig. 9—Effect of aluminum powder on the modulus of rupture and compressive strength (modified cube) of grouted gravel concrete

The aluminum powder was of the polished type. The mold was a 5-gang steel mold regularly used for making 7 by 7 by 38-in. beams of plastic or semi-plastic machine-mixed concrete. All joints of each compartment were carefully sealed with a rosin-paraffin mixture to prevent leakage. The gravel was grouted by depositing the grout from the flow cone in a continuous stream beginning at one end of the beam mold and moving toward the other end as each portion was filled. This operation caused the excess water to drain toward the unfilled end. No attempt was made to compact the grouted gravel

The grouted concrete in the completed beam specimen contained between 4.8 to 5 sacks cement per cu. yd. All beams were cured moist (curing was 2 days in covered molds, 5 days in moist room) and tested damp at the age of 7 days.

In testing, the load was applied at third points to top of beam as cast through flat 2-in. wide bearing plates embedded in gypsum. After testing for modulus of rupture the broken "halves" of the beams were turned on side and tested for compressive strength as 7-in. modified cubes.

Each curve for dry and soaked gravel is based on test of 2 rounds of 5 beams each, made several weeks apart

MISCELLANEOUS TESTS WITH ALUMINUM POWDER

A number of miscellaneous tests were made in connection with a more extensive investigation of the influence of aluminum powder on bond resistance between concrete and steel reinforcing bars, which are of interest in the present study.

Tests with aluminum powder interground with cement clinker

It will be recalled that the tests represented in Fig. 2 were made with four different types of aluminum powder varying in the relative size and thickness of the flakes and in the polish of the surface. Notwithstanding these substantial differences the relation between aluminum content and the compressive strength and unit weight of plastic concrete was practically the same for the four types of powder. It will also be recalled from the notes under "General Description of Tests" that the desired quantity

of aluminum powder for a given batch was premixed by hand with fine sand (0-No. 100 sieve) and added directly to the dry aggregate and cement in the mixer. However, in some supplementary tests, the desired quantity of aluminum powder (polished type) was ground with cement clinker to produce cement having a specific surface of 2200 sq. cm. per gram. For comparison the same clinker was also ground to the same specific surface without aluminum powder but the desired amount was added at the mixer in the usual manner. In these tests neither the compressive strength, unit weight nor the time-swelling characteristics of the concrete was altered by grinding the aluminum powder with the clinker instead of adding it to the batch at the mixer.

Tests with different cements

In another series of auxiliary tests aluminum powder of the polished type was used with 12 different commercial cements varying in alkali content from about .23 to 1.60 per cent. These cements were used with sand and gravel graded 0-1 in. to make plastic concrete having a cement content of 5 sacks per cu. yd. and a slump of 5-6 in. This concrete will be recognized as being of the same type as used in most of the tests reported herein except that the W/C ratio was not constant at 0.63 but varied slightly above or below this value to keep the slump with the different cements constant at 5-6 in. With these 12 cements the average reduction in the 7-day compressive strength of moist cured 6 by 12-in. cylinders was 18 per cent with the addition of 2 grams of aluminum powder or about 9 per cent per gram. The greatest reduction in compressive strength was 22 per cent and the least was 13 per cent. These tests provide further evidence that the strength of plastic concrete made into standard 6 by 12-in. cylinders is generally reduced when the expansions of the concrete is not artificially restrained.

CONCLUDING REMARKS

The preceding discussion and results of tests give a general idea of the various factors influencing the compressive strength of concrete containing aluminum powder. These tests demonstrated repeatedly that the compressive strength of plastic concrete made into standard 6 by 12-in. cylinders was reduced progressively with added increments of aluminum powder so long as no restraint to expansion was imposed at the top of the cylinder. However, as soon as the normal expansion was restrained the relation between aluminum content and compressive strength was altered in varying degrees depending upon the amount of restraint imposed at the top of the cylinder.

The restraint offered by the mold was demonstrated by tests of cylinder specimens of different size made from the same batch of concrete. Without aluminum powder the same compressive strength was indicated by

each size but with aluminum powder the compressive strength showed a progressive increase as the size of the specimens was decreased from the standard 6 by 12-in. cylinder to 4 by 8 and 3 by 6-in. cylinders. Since the smaller molds provided more surface at the interior of the mold per unit of volume of concrete than the larger molds, they can be expected to offer a greater degree of adhesional and frictional resistance to expansion when aluminum powder is present in the concrete. This factor appears to account for the fact that concrete molded into small specimens was definitely stronger and had a denser paste structure than corresponding large specimens from the same batch.

The substantial influence of the different factors reported herein indicate that unusual care must be exercised in testing and interpreting the effects of admixtures of aluminum powder on the unit weight, strength, modulus of elasticity, durability and other properties of concrete. It is of utmost importance that test specimens represent as faithfully as possible the various conditions under which concrete is placed on the job. This is because the degree of restraint varies from practically nothing for pavement slabs to appreciable values in narrow vertical forms in reinforced concrete and mass concrete where both the resistance of the mold and the weight of the superimposed concrete introduce restraints of various magnitudes at different levels.

Tests made in this and other laboratories have shown that under proper conditions of use an admixture of aluminum powder may be very beneficial in improving the bond resistance of horizontal reinforcing bars in deep beams, in improving the effectiveness of grout in filling joints, bedding of heavy objects, or in improving the homogeneity of grouted concrete. In no case should aluminum powder be advocated to overcome deficiencies in concrete mixes that are initially of poor quality.

Discussion of this paper should reach the ACI Secretary in triplicate by April 1, 1943, for publication in the JOURNAL for June, 1943

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Safe Loads for Existing Concrete Bridges*

By F. W. PANHORST†
Member American Concrete Institute

FOREWORD

Committee 320, Safe Loads for Existing Concrete Bridges was organized (with the author of this paper as chairman) to formulate a procedure for determining the safe loads on existing concrete bridges. The assignment is not an easy one. This paper discusses the committee's understanding of the assignment and offers a tentative procedure. The problem is complex; it is more difficult to obtain a definite answer than many realize who have been concerned only with design practice. It is impractical to assign definite values to a great number of the unknown and variable factors involved and recommendations must be based on experience, judgment, and perhaps opinion, to a large extent. The prime purpose of the paper is to elicit discussion, to assist the Committee in making its final recommendations.

INTRODUCTION

There is definite need for a standard policy to be followed in determining the safe strength of existing concrete bridges. There are many old concrete bridges throughout the length and breadth of the country, built previous to modern design practice and without the benefit of modern construction methods and materials. The weights of vehicles now using the highways are far in excess of those contemplated when these bridges were constructed. If all such bridges were rated according to design specifications, modern traffic on the highways would be paralyzed, yet we know from actual experience that whatever the calculated stresses may be, these old bridges are carrying present heavy loads without damage except in extreme cases of poor construction. A questionnaire was recently sent to all of the State bridge engineers asking them for the number of concrete bridges within their knowledge that failed because of washout, faulty design and construction, and overload. Only some three or four, out of 46 that answered, believed they could attribute

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failure to overload and the only positive cases were failure of a very poorly designed and constructed floor slab.

Because of the present military situation which has accelerated the increase in weight of highway vehicles on all classes of highways, the increasing importance of commercial trucking, and the ever-present difficulty of financing the reconstruction of bridges with the additional handicap caused by lack of construction materials at this time, there is definite need for the immediate adoption of rules by which to determine the safe carrying capacity of highway bridges. Military and commercial loadings cannot be accurately measured in terms of the usual "H" loading used for design and to say that a bridge is safe for some standard of loading is not only vague, but often misleading. To be efficient, the particular type of vehicle and the conditions under which it will operate should be specified.

The rated capacity of a bridge should be such that the safety of the traveling public is protected and, at the same time, truck hauling is not penalized any more than is necessary to provide a reasonable degree of safety. Therefore, the ultimate purpose of the work of this Committee is to arrive at a method of establishing the load carrying capacity of any concrete bridge that will not endanger the traveling public, and, at the same time, will allow the heaviest possible truck loading to operate over the highway.

Most discussions have been concerned with design procedure and design working stresses. The problem is not how to design a bridge properly, but to determine how much the bridge can actually carry with safety under given axle weights and spacings and specified operation of the vehicle. Failures from light design being negligible indicates that our methods of concrete bridge design are on the conservative side. Many of the old bridges were designed according to the old 16,000 lb.-650 lb. working stress bases but often very poorly constructed. Experience of many, involving inspections and stress analyses of concrete bridges, shows they can carry loads far in excess of calculated design loads without any signs of incipient failure.

Factors of safety are used to cover many uncertainties such as variations in the strength of materials, unanticipated loads, faulty construction or approximate design rules, etc. Dead load is a relatively definite factor, and the use of a higher working stress for dead than for live load seems justifiable. Under the present design procedure a large, although very variable amount of load-carrying capacity, is provided to take care of over-load, increasing as the spans get longer because the dead load stress of such concrete bridges is usually large in comparison to that caused by the design live load. This is one of several reasons why many

of the substandard bridges can carry present day loadings without signs of distress.

The maximum working stresses and the methods of computing distribution of stress produced by a given weight and spacing of vehicle axles, is what must be agreed on. However, until more definite information is acquired through actual full sized test loads, there will be many factors about which we know very little and, therefore, are open to discussion. All specifications governing the rating of bridges naturally apply to optimum conditions and are to be modified in accordance with the results of periodic and proper physical inspections of the bridge in question.

To sum up the situation, it is apparent that:

(1) There is a definite need for "specifications" by which to determine the safe load capacity of existing concrete bridges, because (a) the present military situation makes it necessary to know load capacity as accurately as possible; (b) a more dependable knowledge of the capacity of a bridge may defer its replacement, thus saving time, man-hours of labor and critical materials.

(2) Experience and more accurate analysis indicate that concrete bridges can carry heavier loads than computations based on the usual design practice of rating them would indicate, thus helping to relieve the transportation situation.

(3) Although general ratings for classifying structures may be based on standard loads such as the "H" type of the American Association of State Highway Officials' Bridge Specifications for Highway Bridges, actual loads should normally be used in computing the stresses, and the rating further governed by the practicability of definitely restricting their operation across the bridge. Due consideration must be given to the probability of passing loads, and also to the effects of frequency of repetition, etc.

(4) The factor of safety under the dead load of a bridge can be less than for the live load, because of the greater certainty as to dead load stresses. This assumes that weights of materials, contemplated additional wearing surface, reduction of section, etc., are properly allowed for otherwise. In rating an existing bridge properly inspected, this is definitely practicable.

(5) The results of past practice should be verified at every opportunity by actual tests, supported by deflection and strain gauge readings, bridges being loaded to complete failure if practicable. Studies of present methods of analyzing all kinds of structural members should be made and all existing test data, records of experience, etc., should be collected and correlated.

(6) Maximum load capacity assumes that periodic inspections have been made by competent, experienced engineers to determine the condi-

tion of all main members, as well as expansion and other important details, and that the rating specifications applying to optimum conditions are modified accordingly. Judgment, based on the result of such inspections, must be a check on the calculated strengths until such time as more information is available to confirm or modify the adopted specifications.

It should be realized, when considering the tentative rating specifications which follow, that concrete bridges do not fail or collapse suddenly and if periodic inspections of doubtful bridges are made with reasonable frequency, as is contemplated, there will be time in which to take proper action. The Committee recognizing the need for something definite as a basis for discussion offers these tentative specifications for the consideration of the members of the Institute.

Proposed Limiting Specifications for Checking the Live Load Capacity of Existing Concrete Bridges

PART A—GENERAL

These specifications shall be used for determining the safe live load capacity of existing concrete bridges. They provide a basis for computing the maximum loads that may be allowed on a bridge when materials are of the best quality, members are acting normally, and deductions in size or area have been made for deteriorated portions. They assume that the bridges are subject to competent inspections as often as the condition of the structures require it, and that the investigating engineer exercises sound judgment in determining the safety factor to be used when the above conditions do not exist. In the determination of proper safety factors, the safety of all types of vehicle loads shall be kept in mind, but the adoption of unreasonably large safety factors, causing unnecessary hardship on economic hauling, shall be avoided.

For all matters not definitely covered by these specifications, the current standard specifications* used for the design of bridges shall be used as a guide. The practice used in designing may be modified, within safe limits, when knowledge of the action of the structural members is sufficiently definite to allow the application of more accurate principles of mechanics.

As a guide in determining where modifications of design practice may be considered, the following facts should be kept in mind:

(*a*) The factors of safety used in designing new bridges provide for a material increase in the future allowable live loading, a variable amount of deterioration, and extreme conditions of long continued loading. The rating of existing bridges applies only for a relatively short time and the loading conditions are more accurately known.

*Of AASHO, or other acceptable specifications—AUTHOR

(*b*) The factors of safety used in rating existing structures must provide only for unbalanced loads, reasonably possible overloads and illegal or careless handling of vehicles varying with the degree of supervision. For either design or rating, factors of safety must provide for lack of accuracy in computing the distribution of the stresses; the possible minimum strength of the materials of individual members, possible differences between the strengths of laboratory tests and the material of the grade used under actual conditions in the structure, and normal defects that may occur in manufacture or fabrication.

The safe vehicle load on a bridge is dependent on the weight, number, and spacing of its axles in relation to the span lengths, the roadway width and the design of the structure itself. Signs at a bridge posted for reduced loads, for practical reasons, can cover only types of load and few restrictions that might apply. The details as to loading and operation of vehicles and the safety factor used in determining its maximum weight will vary with the particular conditions, such as signing a bridge for unrestricted operation of vehicles of each type, operating a particular vehicle under special permit, "H" load ratings for general classifications, etc.

PART B—LOADINGS

1. Dead loads

The dead load of the structure shall be computed in accordance with the conditions existing at the time of analysis or changes contemplated in the immediate future. Minimum unit weights of materials to be used in computing the dead load stresses shall be as follows:

Material	Weight in Lbs. Per Cu. Ft.
Asphalt Surfacing	144
Concrete	144
Steel	490
Cast Iron	450
Earth (compacted)	120
Timber	40

2. Live loads

In computing stresses or allowable loads, one unit only shall be considered in any one lane. The possibility of there being two or more loads following each other in the same lane shall not be considered. The possibility of two units of maximum or equal weights in adjacent lanes and headed in the same direction, in position to produce maximum stresses shall be considered as follows:

Members	Lanes Loaded
Deck	One Lane
Longitudinal beams	One Lane
Floorbeams or Caps	One or Two Lanes
Columns	Two Lanes

When the roadway width between curbs is 16 ft. or less, the above rules regarding one or two lane loading shall not apply, but the stresses throughout the structure shall be computed for one lane loading only.

The gauge distance between wheels shall be taken as 6 ft. The spacing, center to center, of adjacent wheels of passing vehicles shall be taken as 3 ft. and the center of the outside wheel of any vehicle shall be considered as applied at a distance not less than 1.5 ft. from the face of curb. For roadway widths of between 16 and 18 ft., each vehicle shall be placed in the center of its traffic lane.

3. Impact

Impact shall be added to the static loads as follows:

30 per cent, for spans up to 25 feet.
For spans between 25 feet and 325 feet, this percentage shall be reduced on a straight line variation from 30 per cent to 0 per cent.

When the addition of impact to the live loads considered would make a slight load restriction necessary, and its omission would make the computed unit stresses equal to, or below, the allowable safe stress, it is permissible to consider the structure safe for legal loads, provided it is signed for restricted speed.

Impact shall not be considered in members below the caps.

4. Lateral loads

Lateral loads due to wind or vehicles shall not be considered except in special cases as determined by the Engineer. In such cases, a maximum wind pressure of 10 pounds per square foot shall be used.

5. Longitudinal loads

Longitudinal loads shall not be considered in determining load restrictions. However, where longitudinal stability is considered inadequate, the structure should be signed for restricted speed.

6. Thermal forces

Stresses set up by thermal forces shall not be considered unless the Engineer, as a result of his inspection, determines that they are especially and unusually important.

PART C—DISTRIBUTION OF LOADS

1. Distribution to longitudinal beams

The fraction of a wheel line load carried by each beam shall be determined as shown below, except that when the spacing of girders, center to center, is 5 ft. or more, it shall be determined by the principles of mechanics, assuming the floor to be hinged over its points of support.

	Fraction of Wheel Load to Each Girder
Concrete slab floors, either with or without sub-floors, greater than 4" thick	$\frac{S}{6.0}$

S = Center to center girder spacing, in feet.

The combined load capacity of the longitudinal beams in any panel shall not be less than the total live and dead load in the panel.

2. Distribution to floorbeams and floor slabs

Distribution of wheel loads to floor beams or concrete slabs shall be computed in accordance with the current design specifications, except in cases beyond the limitations of the design specifications, when computation of stresses will follow the principles of slab and flexible beam analysis.

PART D—ALLOWABLE MAXIMUM UNIT STRESSES

1. General

The maximum allowable live loads on any structure shall not be greater than those which, combined with all other loads, will produce the maximum unit stresses set forth hereinafter. These maximum basic unit stresses shall be used only when, in the judgment of the Engineer, the materials under consideration are sound and practically equivalent in strength to new materials of the grade and qualities that would be used in first class construction.

The effective area of members to be used in the calculations shall be the gross area less that portion which has deteriorated due to decay or corrosion. This net area shall be determined by the Engineer in the field.

Span length for the determination of stress shall be assumed as follows:

	Span
Beams and Slabs:	
(*a*) Freely supported	C/C supports but not to exceed clear span plus depth.
(*b*) Restrained or Monolithic	Clear span.

2. Allowable (working) stresses

(*a*) Stress in steel, bending,

$$\frac{DL \text{ (per cent)} \times 30{,}000 \text{ psi} + LL \text{ (per cent)} \times 27{,}000 \text{ psi.}}{100}$$

(*b*) Maximum moment in concrete beams, (p not greater than about 0.025). $M = (1 - 10p)f_s A_s d$

Where, f_s = allowable stress in steel as in (*a*)

A_s = area of steel reinforcement for tension.

d = depth of beam above center of reinforcement.

$p = A_s/bd$.

b = width of beam.

The above also applies to T-beams when the flange thickness, t, is greater than $0.537d$, otherwise according to the following formula: $M = f_s A_s(d - t/2)$

(*c*) Stress in concrete, bending. Not a critical factor for ordinary values of p. In special cases the value given in the design specifications increased by 50 per cent may be used as a basis.

(*d*) Maximum shear: maximum stress in steel shall be as given in (*a*); Maximum stress in the concrete shall be based on values given in the design specifications increased by 50 per cent.

(*e*) Maximum allowable axial load in columns (tied),

$$P = 0.27f_c'A_g + 1.2A_sf_s$$

A_g, gross area of column.

Long columns having l/d greater than 12 shall have their axial load reduced in accordance with the formula,

$$P' = P(1.3 - 0.03\ l/d)$$

In using the design values for (*c*), (*d*), or (*e*), if the value of f'_c is not known, a maximum value of 3000 psi. shall be used.

When the amount of tension steel in beams cannot be ascertained, a value of $p = 0.008$ may be assumed.

When the stresses in a concrete structure exceed those computed in accordance with the above but the structure shows no visible signs of distress, or, when the computed stresses are less than the allowable and signs of incipient failure are present, the structure shall not be rated for restricted loading but shall be inspected at frequent intervals.

When the computed stresses in a concrete structure exceed those allowed above and the structure also shows signs of incipient failure, the load shall be restricted.

PART E—RESTRICTION OF LOADS

The total unit stress caused by Dead Load, Live Load and such other Loads as are deemed applicable to the structure shall not exceed the allowable unit stress as set forth in these specifications or as reduced by the Engineer from his inspections. When it becomes necessary to reduce the allowable live loads on the structure, such reduction shall be based on the assumption that each axle load has a constant relation to the total load of the vehicle or vehicle combination.

Discussion of this paper should reach the ACI Secretary in triplicate by April 1, 1943, for publication in the JOURNAL for June, 1943

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Discussion of a paper by F. W. Panhorst:

Safe Loads for Existing Concrete Bridges*

DISCUSSION BY CLYDE T. MORRIS, HOMER M. HADLEY, G. F. BURCH, and AUTHOR

By CLYDE T. MORRIS†

The paper is very timely. Calculations for the ultimate strength of members and the use of a factor of safety are especially applicable when rating an existing structure. The paper "Balanced Design for Reinforced Concrete." by A. J. Boase and C. E. Morgan, in the February JOURNAL also makes use of a factor of safety.

A factor of safety is the ratio between the ultimate strength of a member and the working load. Its use is contingent upon a fairly accurate knowledge of the ultimate strength.

It is well known that the usual straight line theory of stress variation in reinforced concrete members in flexure does not give results in accord with tests to failure. This was well brought out in the paper, "Design of Reinforced Concrete Members under Flexure or Combined Flexure and Direct Compression,"‡ by Chas. S. Whitney.

Tests show that beams of concrete whose cylinder strength is 2500 psi or greater, and reinforced with less than about 2½ per cent of steel in the tension side, fail when the yield point of the steel is reached and not by crushing of the concrete. Mr. Panhorst recognizes this in his proposed formula for bending moment in concrete beams, p. 191.

$$M = (1 - 10p) f_s A_s d \quad \ldots\ldots (1)$$

According to present day practice, the steel in the tension face rarely exceeds 2 per cent.

In Table 2, of Whitney's paper, p. 492, are given the results of tests of 24 beams with reinforcing varying from .49 per cent to 1.96 per cent and concrete whose cylinder strengths varied from 1555 to 5589 psi. In these tests, the steel lever arm, usually denoted by jd, averaged about $.91d$

*ACI JOURNAL, Jan. 1943; *Proceedings* V. 39, p. 185.
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‡ACI JOURNAL, Mar.-April 1937; *Proceedings* V. 33, p. 483.

which is considerably greater than $(1 - 10p)$, as given by Mr. Panhorst's formula. The following formula for j agrees surprisingly well with the tests given in Whitney's Table 2.

$$j = 1 - p\left(13 - \frac{f'_c}{600}\right) \quad \ldots\ldots (2)$$

in which $p = A_s/bd$

f'_c = cylinder strength of the concrete in psi.

Then $M = f_y A_s jd$ ..... (3)

in which M = bending moment at failure

f_y = yield point strength of the steel.

Any existing bridge which is being investigated probably would be several years old. Ordinarily, concrete will increase in strength from 40 per cent to 60 per cent in the period from 28 days to several years. If the concrete cylinder strength at 28 days was 2400 psi, it should be at least 3600 psi at the time of rating. This would give us a value of $j = (1 - 7\,p)$. If the 28 day strength were 3000 psi the cylinder strength at rating would be about 4500 psi, which would give a value of $j = (1 - 5.5\,p)$.

In rating an old bridge, if test data concerning the strength of the concrete at the time of its construction are not available, a competent inspector can usually estimate the present strength with sufficient accuracy for rating purposes unless the percentage of reinforcing is high and there is danger of failure by crushing the concrete. In that case it frequently is possible to cut a prism or core of the concrete from some unimportant part of the structure, refill the hole with new concrete, and test. A prism 6 x 6 x 12 in. makes a very satisfactory test specimen.

For beams with sufficient reinforcing so that failure of the concrete will take place before the yield point in the steel is reached, Whitney* gives the following formula for ultimate strength.

$$M_c = \frac{f'_c bd^2}{3} \quad \ldots\ldots (4)$$

in which M_c = the bending moment at failure.

The rating of simple beams or slabs is comparatively easy.

In columns, theoretically subject to axial compression without bending, a larger factor of safety should be used because of uncertainties involved. Whitney* gives the following formula for the ultimate strength of a symmetrically reinforced member under concentric load.

$$P = 0.85\,f'_c A_g + f_y A_s \quad \ldots\ldots (5)$$

in which A_g = gross area of the concrete,

A_s = total area of steel in both faces.

In members subject to combined flexure and direct compression, the fiber stress due to direct compression may be computed by transforming equation (5).

*Previous reference.

$$f_c = \frac{P}{A_g + n'A_s} \quad \ldots\ldots (6)$$

in which f_c = fiber stress due to direct compression

$$n' = \frac{f_y}{.85f'_c} \text{(usually } n' = \text{about 12 or 15)}$$

For a symmetrically reinforced member which fails by crushing the concrete, the fiber stress in bending may be computed from equation (4) by adding a term representing the steel in the compression face.

$$M_c = \frac{f'_c bd^2}{3} + \frac{f_y A_s d'}{2} \quad \ldots\ldots (7)$$

in which d' = distance between the steel in the two faces

$$\text{Then } f_B = \frac{6M}{2\,bd^2 + 2.55n'\,A_s d'} \quad \ldots\ldots (8)$$

in which f_B = fiber stress in the concrete due to flexure.
The total maximum fiber stress should not exceed $.85\,f'_c$, or

$$f_c + f_B = .85\,f'_c \quad \ldots\ldots (9)$$

In rating an existing bridge a factor of safety of one for the dead load and 2 for the live load and impact should be safe.

By HOMER M. HADLEY*

This is a notable paper. It is, so far as the writer knows, the first formal recognition ever to be given to the fact that concrete bridges, as they have been designed for many years, can carry loads greatly in excess of—in fact, out of all reasonable relationship to—their supposed capacities. The committee's questionnaire,† sent to all State bridge engineers and responded to by 46 out of 48, elicits the fact that only 3 or 4 "believed they could attribute failure to overload and the only positive cases were failure of a very poorly designed and constructed floor slab." The author calls attention to the fact that there are many old concrete bridges throughout the country carrying loads far in excess of those for which they were designed. "If all such bridges were rated according to design specifications, modern traffic on the highways would be paralyzed, yet we know from actual experience that whatever the calculated stresses may be, these old bridges are carrying present heavy loads without damage except in extreme cases of poor construction." It is indeed a valuable service which the Committee renders the bridge building profession in thus focussing attention on the difference between theory and reality. Theory needs to be radically revised if it is to avoid appearing fatuous and silly. The lay mind is apt to ask, particularly if times grow

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hard and money scarce, what sense there is in such inordinate strength, what kind of engineering skill and judgment is it that requires the use of expensive materials to this extent? True, concrete bridges are thereby rendered immune to practically any overload but it is likewise true that after a reasonable margin of safety is provided still additional safety is a costly luxury that can be ill-afforded by anyone.

As further "fuel to the flames," so to speak, the following cases of bridge loading are cited. They are not new but may well be recalled to attention. Details regarding them are readily available to everyone.

The load test of the arch bridge across the Yadkin River, North Carolina, was described in complete detail in *Public Roads*, Dec. 1928. The span subjected to heaviest loading was a 2-rib arch span of 146 ft., 3 in., 28 ft., 3 in. rise, 17-ft. roadway, build of 1:2:4 concrete. To destroy all strengthening effects of the roadway upon the arch ribs the deck was completely cut through at every panel point. For maximum loading two large water tanks 12 ft., 6 in. by 20 ft., 0 in., by 18 ft. high, were placed on each side of the span in the most unfavorable position and were filled to produce a load, tanks and contents, of 320 tons. Result: arch ribs deformed somewhat to produce maximum deflections of about ¾ in., up and down.

At Glendale, California in 1936, army flood-control work necessitated the removal of a recently-built concrete rigid frame bridge of 43-ft. span at Central avenue. It had two 5-ft. sidewalks and 50 ft. of roadway. A load test was arranged. Under maximum load, steel billets in a wall 8 or 9 ft. high—430 tons of steel billets—were piled transversely across the bridge at midspan. A deflection of ⅜ in. was measured. These 430 tons of load caused no failure in the bridge—merely a ⅜-in. deflection. Maximum measured unit stress in reinforcement, 15000 psi. See *Engineering News-Record* Nov. 17, 1938 p. 615 or *Southwest Builder and Contractor*, Aug. 21, 1936. Fig. A shows billets and bridge, in conjunction; Fig. B, the bridge.

The load test whereby 115 tons were applied to the westerly 140-ft. span of the Purdy Spit Bridge, Pierce County, Washington, is described in considerable detail in *Engineering News-Record* Mar. 3, 1938 p. 339. The results closely resembled those which attended the load testing of the balcony of a new vaudeville theatre in a Pacific Coast city a now-considerable number of years ago. The framing in this case was structural steel. When the theatre was opened to business, full-page advertisements in all the city's newspapers announced the fact. The wonders of this new playhouse were described in lavishly laudatory terms. The balcony, built without columns beneath that could obstruct anyone's view, was a feature receiving highly honorable mention. The advertisements stated that this balcony had been most carefully and painstakingly designed by

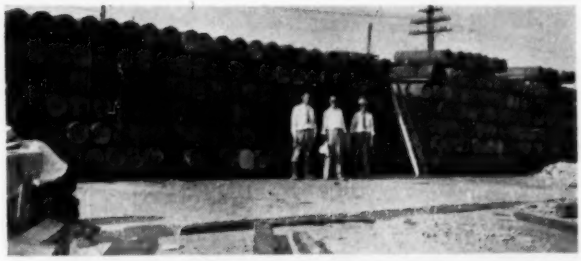

Fig. A — Central Avenue Bridge, Glendale, Calif. 430 tons of steel billets applied on transverse centerline

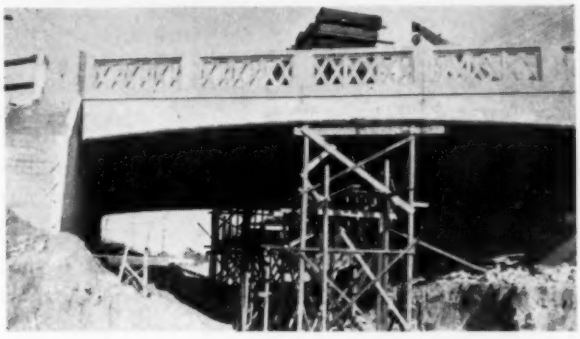

Fig. B — Central Avenue Bridge — Midspan scaffolding and gage supports. Here, at midspan, 3/8-in. deflection occurred under 430-ton concentrated load

the architect, but that to make assurance of its safety doubly sure he had ordered a test load of a prodigious number of tons placed upon it. The city's building department closely cooperated and supervised. "Under this load," said the advertisement, "the balcony deflected precisely one hair's breadth—exactly what Architect had anticipated!"

Another little job offering somewhat ironic testimony on concrete bridge design is the pavement on the bascule of the Burnside street bridge in Portland, Ore. The Williamette river bridges in Portland are taxed with traffic day and night and are subjected to as heavy wheel loads as bridges receive anywhere. The floor of this bascule is a 2-way reinforced slab, of 1:1:2 concrete, 4¾-in. thick, panels being as large as 5-ft. by 5 ft., 8 in. Punching shear is a dread thought to many but be that as it may, these slabs have never "punched." This deck is described in *Engineering News-Record* May 13, 1937 p. 710.

Still another case is the present belt line, former main line, bridge of the Union Pacific Railway at Riverside, Calif. See Fig. C. This is a skewed flat slab structure, 58 ft. square across from face to face of abutments. This 58-ft. perpendicular distance is divided by the columns at the roadway margins into spaces 11 ft., 6 in., 35 ft., 0 in., 11 ft., 6 in. The slab thickness varies, averaging 22 in. at midspan, 20 in. at the columns. It is to be realized that the top of the structural slab in Fig. C is at the bottom of the clearance sign and that what one sees above this

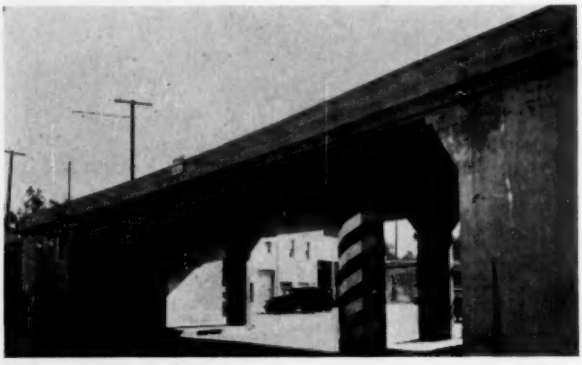

Fig. C — Former main line, present beltline, bridge of Union Pacific Railway at Riverside, Calif.

level is the curb for retaining ballast. The bridge has a certain appearance of unreality when viewed as a former railway main line and present belt line structure in daily use. Thoughts of the sober, serious business it ordinarily is for railways to negotiate a crossing such as this add to the incredulity. Nevertheless there it stands! *Engineering News-Record* Dec. 2, 1937 p. 894 gives fuller details.

The author states that it is the ultimate purpose of the Committee's work to arrive at a method of establishing the load-carrying capacity of any concrete bridge; apparently it believes that maximum premissible loads should be "computed." It is to be recognized, of course, that bridge building is largely a public enterprise and that elaborate calculations are an established means of impressing the clients with the esoteric mysteries of bridge design. There are other means of acquiring knowledge of load capacity than by mathematical calculation. The latter method does in fact stand rather grievously indicted by the introduction to the specification being considered by the Committee, which indicates that *computed* capacities fall short—indeed far short—of actual capacities. The proposal is therefore advanced that the desired information regarding reasonably safe permissible loading be obtained directly and positively by load testing, not to destruction of course but to an extent markedly in excess of what may be anticipated will be maximum loads in service. If the critical parts of a bridge successfully carry a certain load, then they carry it, anyone's doubts and misgivings to the contrary notwithstanding.

It is the distinctive characteristic of reinforced concrete that it forms into an integral, monolithic structure. It is the distinctive, physical characteristic of integral structure that deformation of a certain portion of the whole is minimized by its surrounding parts and these surrounding parts, thereby being deformed, are themselves strengthened by and trans-

fer deformations to their adjacent parts, and so on. In a concrete bridge deck, the amount and distribution of reinforcement being average and not abnormal, the most severe stresses develop at the points where heavy concentrated loads are applied and it is the ability of the concrete deck to distribute load concentrations broadly and to render wide portions of the deck effective in resisting main span stresses that is its great advanatge. If therefore it is established that the bridge deck in question can sustain the local concentrations of loads to which it may be subjected, the uncertainties of the case appear to be almost wholly removed, for beyond this there is practically no question concerning main span stresses—see the results of Committee 320's questionnaire. As the heavy live load concentration points are departed from, dead load assumes a position in the field of total stress of increasing importance while live load is increasingly minor. In long and heavy spans the fact that the bridge stands and carries itself is quite conclusive evidence that it can carry all the vehicular loads that will come upon it.

There can be no question that the roadway slab, particularly the slab used on short spans between beams or stringers, is the lightest, thinnest structural part of a concrete deck. It is subjected to heaviest load concentrations, has the least opportunity to spread and distribute its load; it is almost invariably the weakest element of the deck assembly. Next in order is the beam or stringer supporting the slab, and so on progressively. Since the only positive cases of failure coming to the Committee's attention were slab failures, and since the next most likely member to fail because of heavy load and limited ability to distribute it, is the slab-supporting member, it should not prove difficult actually to apply loads to slabs and beams greatly in excess of anything which can possibly come upon them. A truck of maximum capacity which could be heavily overloaded might be assigned for a time to the particular task of testing concrete bridges and could be suitably equipped for the purpose. It could be readily shifted from point to point on the deck and by a suitable arrangement for jacking against its rear axle, concentrated loads double its rear wheel loads could be applied to typical and to all questionable points in slabs and beams. By limiting the jack's bearing area upon the slab, extremely high unit bearing and unit shears can be obtained. From proven ability to support loads of such definitely known great magnitude, it can then either be assumed directly that those lesser loads, the ordinarily-to-be-expected maxima, can successfully be sustained, or with due regard for the amenities, that fact can be computed by statistical mechanics and can be checked by other appropriate mathematical means. At all events this method of load determination by actual testing is suggested as one dependent on no assumptions for its validity.

In closing this discussion the writer would again express his belief in the high value of the work undertaken by Committee 320. The chairman's paper is refreshingly sane and gumptional.

By G. F. BURCH*

The introduction states that the ultimate purpose of the work of Committee 320 is to arrive at a method of establishing the load carrying capacity of any concrete bridge, allowing the heaviest possible truck loading to operate over the bridge without endangering the traveling public. Elsewhere, two different ideas are expressed as to the meaning of "the heaviest possible truck loading." One idea is expressed in the statement, "To be efficient, the particular type of vehicle and the conditions under which it will operate should be specified." The other idea is expressed in such terms as "the rated capacity of a bridge," and "general ratings for classifying structures." I assume that the proposed Limiting Specifications are intended to cover both the case of a definite single load crossing the bridge under specified conditions of speed, wind, position with respect to traffic lanes or center line of roadway and distance from other loads, and also the loading that might be stated on a posted sign as the maximum weight of vehicle that could cross, without regard to definite features of the load or other conditions.

In computing the capacity of a bridge for a definite single load, it may be satisfactory to omit such forces as wind, and impact under certain conditions, and to make other assumptions compatible with the restrictions imposed or the particular conditions in the case. Separate computations would be required for each separate questionable load in such cases.

The specifications as written, however, seem to refer as well to the general maximum safe load. This is indicated by the requirements for considering only one unit in one lane, the statements as to when loads in adjacent lanes should be considered, the distance between wheels of passing vehicles to be assumed, etc. In such cases, I believe the capacity of a bridge should be computed by considering all the loads, such as wind, temperature effect, etc., considered in the design of a new structure, and assigning the same values to them as for design. The only difference between the rating of an existing bridge for general loads and the design of a new structure is that the design of a new structure should provide an additional factor of safety for two things not considered in the rating of an existing bridge. There are: The possibility of future changes in traffic, causing increased stresses, and allowance for future deterioration of the materials. Since these two factors need not be considered in

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determining the present capacity of existing bridges, I believe that an allowable unit compressive stress in concrete due to bending, of 1500 psi in undamaged masonry, and a unit tensile stress in steel of 24000 psi of sound metal, might well be used in the rating of existing bridges. Other stresses such as in columns, bond, shear or diagonal tension, etc., 50 per cent greater than those used for designing new structures might also be used for material showing no deterioration. Except for the increases in allowable unit stresses on account of the two factors mentioned, I believe the specifications for determining the capacity of existing bridges for general loads and those for the design of new structures should be the same.

AUTHOR'S CLOSURE

The writer acknowledges discussions by Professor Morris, Mr. Hadley and Mr. Burch, and believes Committee 320 will find them to be of definite value in obtaining a final solution of the problem. One or two points in the discussions seem open to some question and this opportunity is taken to comment on them further.

Mr. Hadley suggests putting a load on every bridge to test its safe load capacity. To be of value, such a test load would have to produce greater stresses in all critical members than any load that is expected to cross the bridge. Loads for which permits are asked seem to to get heavier and heavier and it is hard to know how heavy a test load will cover all possibilities. Of course, if the test load is sufficient to break down or damage the bridge, that is a positive answer so far as maximum load is concerned but it is a very expensive way to find out. A general test of all concrete bridges, even in a limited way, would involve considerable time and expense. There are about 2,300 concrete bridges on the State Highway System in California and to test them in this manner would cost something on the order of $100,000, even if carried through in one operation. It would be hard to convince finance-conscious highway officials that such an expenditure is economically desirable, since it benefits only a very small per cent of the traffic. It, apparently, would take a long time to do that much damage to the existing bridges, even under the most liberal interpretation of allowable strength, based on theoretical analysis supported by knowledge gained through observation and experience which is the present practice. For the present at least, it would seem that this is the only practical procedure.

Furthermore, a single load that is heavy enough to cause visible damage to a concrete bridge is not a proper criterion for maximum load. Nor is the fact that no immediate visible damage is done any assurance that the bridge is not suffering damage that will materially shorten its service life. Such failures as have taken place are gradual,—sometimes

no visible damage is apparent for many years after the bridge is opened to traffic and finally occurs with no increase in the weight of loads to which it has been subjected. Perhaps temperature change or shrinkage is involved. More thorough knowledge of actual stresses and their causes is needed and there can therefore be no argument about the need for comprehensive studies and tests of typical structures—to destruction when it is practicable. This need expressed by the Committee will be found on p. 187[5] of the paper. The opportunities for making such tests should be many in the ordinary course of normal construction but it is necessary that the work and the results be properly correlated.

Professor Morris questions the accuracy of the proposed formula for bending moment in beams: $M = (1 - 10p)f_s A_s d$. The value, 10, can be arrived at by assuming values of 34,000 psi for f_s and 2,000 psi for f'_c, in Whitney's formula, $\frac{m}{2} = \frac{f_s}{.85f'_c}$, which is contained in his formula for c/d (analogous to j in the "straight line" theory): $\frac{c}{d} = 1 - \frac{pm}{2}$. It is seen that the value of c/d for $p = .01$ (about the value commonly found in older designs), would be 0.90. This seems to be a reasonably conservative value. Some suggestions were made previously that a value $c/d = 0.90$ for all cases would be close enough, considering the unknowns connected with rating an existing structural member as to its resisting moment. However, the value of p can, nearly always, be found from the plans or by investigation, and its effect is too important to be included in a general value for c/d. If it had been considered practical to estimate the value of f'_c, it also could have been taken into account by assuming a value of about 34,000 psi for f_s of structural steel and get a factor $\frac{20{,}000}{f'_c}$ to be used in place of the constant 10.

Professor Morris' suggestion that the value of f'_c can be determined by taking cores from the member is open to serious argument. First of all it does not seem economically practicable in the case of the ordinary small highway structures. Furthermore, cores from adjacent sections are quite likely to have very different strength properties and the relation between the breaking strength of such cores as compared to standard test cylinders is not too well established. The cores taken from the ribs of a reinforced concrete arch bridge on a main city street in California 22 years after its completion, showed breaking strengths ranging from 1100 to 2200 psi (corrected for shape). Five other cores were in too poor a condition to be tested. On the basis of these tests the computed safe load on the bridge was far below the legal limits but, nevertheless, it has

carried the heavy traffic of a business district of a large city for 30 years without signs of distress.

The strength of columns was omitted from the first draft of the proposed specifications because of the fact in bridge construction it rarely if ever is a critical factor in determining the maximum safe load capacity. It was felt that rating specifications, unlike design specifications, need cover the limits to be placed only on important and reasonably definite strength factors that govern the safe load capacity.

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The Microscopic Structure of Hydrated Portland Cement*

By L. T. BROWNMILLER†

SYNOPSIS

Microphotographs illustrate the structure of neat hydrated portland cements as seen in reflected light. They show that a considerable amount of unhydrated cement remains in type 1 and type 2 cements after hydration for 28 days. Type 3 cements show much smaller percentages of unhydrated material even at earlier ages.

Most of the principal constituents of the original clinker can be recognized in the unydrated fractions. The photographs give no evidence that any major constituent of the cement is selectively or completely hydrated at any age. The rate of hydration depends more specifically on the surface exposed to the action of the water than on the chemical constitution.

The effects of laitance formation are shown by illustrations of the difference in particle size distribution in the laitance as compared to that within the main body of the cement.

Other photographs show the size, amount and distribution of the $Ca(OH)_2$ which is liberated during the hydration processes. It is estimated that about 15 per cent of $Ca(OH)_2$ has formed at 28 days in the cements examined. The polishing and etching technique described could be applied readily for the preparation of specimens for accurate measurement of the $Ca(OH)_2$ by means of mechanical devices such as a Wentworth micrometer.

The final photographs show some detail of the structure of the hydrates other than the $Ca(OH)_2$. That structure is extremely complicated, but a further development of microscopic technique should be useful in solving some of the riddles which confront cement technicians in attempting to evaluate cements on the basis of performance in concrete.

INTRODUCTION

This paper presents results of some microscopical examinations of hydrated portland cements. These examinations were made by the reflected light method commonly used for determining the microstructure of metals and alloys.

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Stein [1] and Wetzel [2] applied that method in their studies of the structure of portland cement clinker a number of years ago. Later Tavasci[3], Insley [4] and others developed polishing and etching technique to a degree which made the adoption of the method by cement research laboratories practical.

The application of the same method to studies of the structure of hydrated cement and concrete was generally neglected until Tavasci [5] published an article on that subject in 1941. In that article the principal features of hydrated cements were illustrated.

The microphotographs of this paper demonstrate some additional details of set cements. The first photographs show the rate of hydration of cements up to 28 days, as measured by a decrease in the amount and size of unhydrated particles with age; they show clearly that the larger particles of the cement are attacked only on the surface by the mixing water, leaving cores in which the principal constituents of the original clinker can be identified readily. These studies gave no evidence that any single, major constituent of the clinker was selectively and completely hydrated at 28 days.

Other photographs show the relative amount and distribution of the $Ca(OH)_2$ which is formed during the hydration processes. A considerable amount of that compound is formed during the reaction of the cement with water. In the cements examined here, the amount of $Ca(OH)_2$ is estimated at about 10-15 per cent of the total. In the laitance, the $Ca(OH)_2$ tends to agglomerate into large crystal aggregates, while within the main body of the cement it is more uniformly distributed in smaller crystals.

Other photographs show some structural details of the hydrates other than the $Ca(OH)_2$. The structure of those hydrates is extremely complicated. In general it has been impossible to differentiate them, although in the laitance some relatively large crystals develop, which may be C_3A-hydrate.*

Experimental technique

The technique of polishing the hydrated cements was varied from the usual procedure for polishing clinker [6], because of the softer nature of the hydrates, e.g. the $Ca(OH)_2$. The most satisfactory sections were obtained by final polishing of the specimens by hand on a chamois with tin oxide. Hand polishing is a laborious procedure, but high speed wheels such as are commonly used for polishing clinker, tend to tear

(1)——See References at end of paper.

*The system of abbreviation used in this text is that common in technical reports on cement constitution; thus

$C_3S = 3CaO.SiO_2$
$C_2S = 2CaO.SiO_2$
$C_3A = 3CaO.Al_2O_3$
$C_4AF = 4CaO.Al_2O_3.Fe_2O_3$

away the hydrates so that the sections are unsatisfactory for examination. Tin oxide is a more suitable polishing media than ferric oxide since it does not smudge the surface as badly as ferric oxide does, and it is more readily cleaned from the finished specimens.

In general the neat cement specimens used for these examinations were tough and dense with relatively few voids. Accordingly no impregnants were used in any of the specimens described. Usually the hand polished surfaces were uniform and even with no evidence of loss of hydrates torn away during the polishing process.

All of the illustrations except Fig. 8, represent neat cements with 40 per cent mixing water. The specimens were molded and sealed in glass vials for 24 hours. Storage conditions after 24 hours are given in the detailed descriptions.

Immediately after removing the specimens from storage for examination, a flat surface was ground on them with the finest grade of corundum. That face was stained with several applications of patent blue dye in alcohol. The dye was absorbed by the cement hydrates but not by the unhydrated particles. The dye stain was used primarily to obtain good contrast between hydrated and unhydrated material for photographical purposes.

After staining, the specimens were allowed to dry for several hours (or over-night) in an oven at 95 F. to check hydration. The stained surface was then polished on a chamois. Sections which were dried in the oven overnight showed no difference from corresponding sections polished immediately after removal from storage. Apparently at 95 F. the surface dries sufficiently to reduce all hydration to a minimum. Specimens prepared in this way may be kept in a desiccator for long periods with no apparent changes in structure.

As will be pointed out later, the structure of the hydrated cements depends on conditions of bleeding, laitance formation, amount of storage water accessible to the cement and other factors. Accordingly in illustrations comparing the rate of hydration of cements at various ages, an attempt was made to photograph fields of each cement at the same position within the specimen. All photographs, except those illustrating laitance formation, were made from fields about 2 in. below the top of the cylindrical specimen and about ¼ in. inside the circumference of the cylinder. In massive concrete where outside water is limited, the conditions of hydration of the cement might be different from those in the small specimens illustrated here.

RESULTS OF EXAMINATIONS

It is well known that the setting and hardening of cements are due to chemical reactions with the mixing water. The reactions progress as the

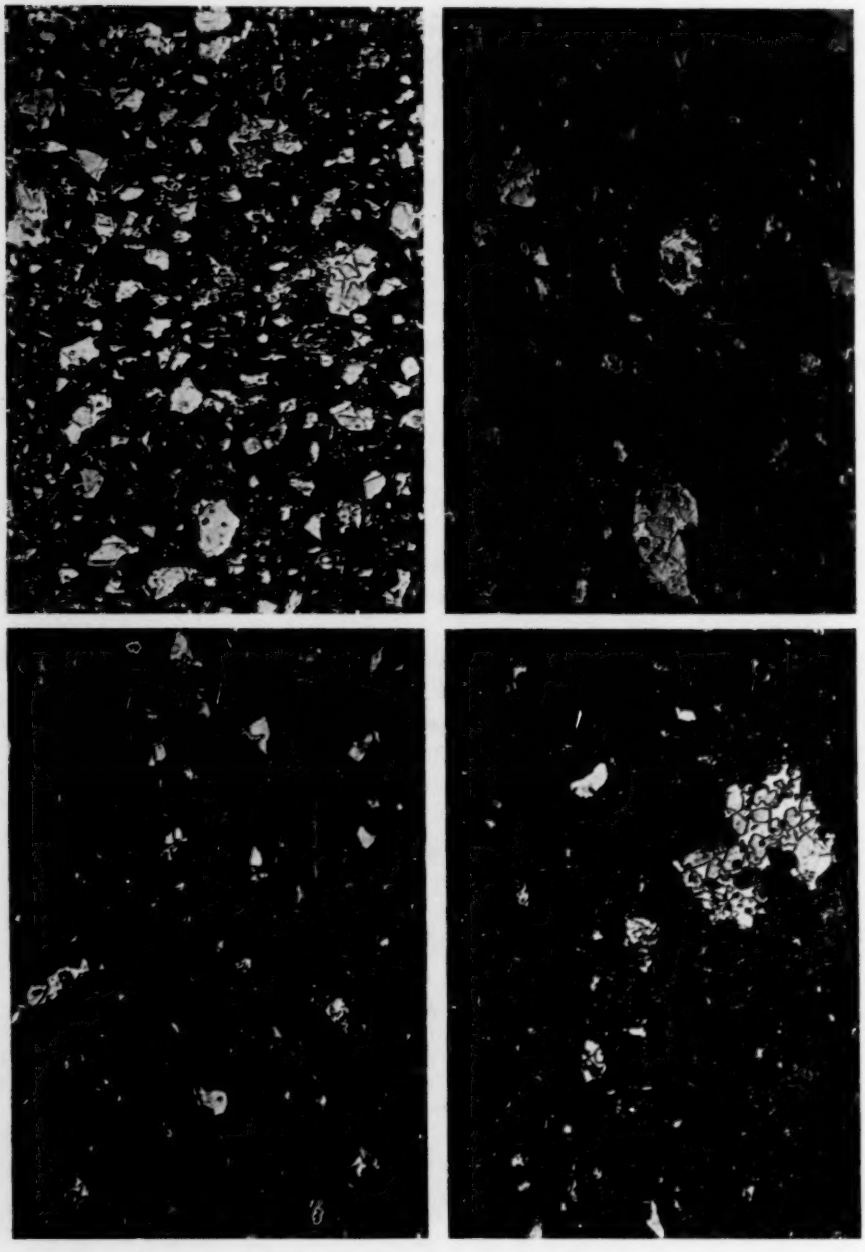

Fig. 1 to 4

Fig. 1 (upper left)—One day hydrate—neat high early strength cement, showing white unhydrated cement particles and dark hydrates; Fig. 2 (upper right)—Three day hydrate—neat high early strength cement; Fig. 3 (lower left)—Seven day hydrate—neat early strength cement; Fig. 4 (lower right)—Seven day hydrate—neat high early strength cement, showing one 90-micron unhydrated particle (Magnifications 220 X)

specimens age. Even after long periods of storage in water, a certain amount of the original cement remains undecomposed.

The first photographs, Fig. 1-8, were selected to illustrate the rate of reaction of cement with water. All of those specimens, except No. 8, were stored in glass vials for one day. After 24 hours the glass was broken away and the 3-, 7-, and 28-day specimens were aged in water.

In the photographs, high contrast between hydrated and unhydrated cement was obtained by staining the specimens deeply with the dye and by overdeveloping the photographic prints. Accordingly in Fig. 1-8 the hydrated cement appears black and the unhydrated particles appear white.

Specimens 1-8 were etched for 3 seconds in a 15 C. solution containing one drop of concentrated HCl in 10 cc alcohol. This acid etchant was used to brimg out some of the details of the unhydrated cement;—specifically to show the outlines of C_3S and C_2S crystals.

Fig. 1, at 220 magnification, illustrates the relative amounts of hydrated and unhydrated cement in a neat high early strength cement after one day vial storage. The cement contained 66 per cent C_3S, 4 per cent C_2S and 14 per cent C_3A by calculation from the oxide analysis. The amount of mixing water was 40 per cent.

From the appearance of Fig. 1, it is estimated that about one-third of the original cement remains unattacked by the mixing water at 1 day. (The estimates of the amounts of unhydrated cement in the photographs are entirely visual. In order to obtain accurate quantitative data it would be necessary to measure a large number of fields by some mechanical means, for example with a Wentworth micrometer, and apply density corrections to the measurements.) The largest particles remaining in this section are about 45 microns in size. Crystals of angular C_3S can be seen in them, particularly in the 45 micron particle at the right center of the photograph.

The surface area of the cement was 2580. Particle size distribution curves showed about 2 per cent of the cement was coarser than 60 microns and about 3 per cent between 50 and 60 microns. The field of Fig. 1 does not show any of the coarsest particles of the cement (see Fig. 2 and 4). It is doubtful if any of the 45 micron particles found in Fig. 1, were coarser than 60 microns originally. From the frequency of occurrence of the 45 micron particles in the one day hydrate, it is assumed here that their average size originally was about 60 microns. If that assumption is correct, then during the first day of hydration of this cement, the 60 micron particles decreased to 45 microns; that is the radius of the particles decreased 7½ microns during the first day.

Fig. 2 is a 220 magnification of the same high early strength cement at three days (1 day in a vial and 2 days in water). The amount of unhydrated material has decreased to about 5 per cent. Thus hydration has progressed in accordance with the theories of increased heat of hydration, strength gains, etc. The largest cement particle remaining in this section (lower center) is about 60 by 30 microns. Undoubtedly that particle was of the 2 per cent fraction originally coarser than 60 microns.

The other larger particles of the photograph are about 30 microns. By comparing Fig. 1 and 2 one may assume that the average larger particles have decreased from 45 microns at 1 day to 30 microns at 3 days. In other words hydration proceeded into the cement particles at the rate of 7½ microns during the 2nd and 3rd day of water storage.

Fig. 3 is a 220 magnification of the same high early strength cement at 7 days (1 day vial storage and 6 days in water). Practically all of the cement has hydrated,—possibly 1 per cent remaining undecomposed. The larger particles which remain are about 20 microns. Comparing Fig. 2 and 3 it is apparent that those particles have decreased from 30 microns at three days to about 20 microns at 7 days. Thus hydration has advanced 5 microns into the particles from 3 to 7 days.

During the first seven days the rate of hydration along the radius of the particles of this particular cement may be estimated as follows:

1st day	7½ microns
2nd and 3rd days	7½ microns
4th to 7th days	5 microns

Summarizing, the rate of hydration of the particles of this particular cement has proceeded into the individual grains at the rate of 20 microns during the first seven days. During that period particles which were originally finer than 40 microns probably are completely hydrated while 60 micron particles may still be 20 microns in size at seven days. These rates of hydration are only given as an example of what apparently occurred in these specimens.

Fig. 4 is another field of the 7-day hydrate of the high early strength cement at 220 magnification. This photograph was included to illustrate that a small amount of oversize material was present in the cement. One coarse particle is visible in this field; that particle is about 70 by 90 microns after 7 days hydration. Crystals of angular C_3S are plainly visible in it. Except for that particle, the general appearance of the 7-day hydrate of Fig. 4 is about the same as that of Fig. 3.

Thus in a finely ground cement of high C_3S and high C_3A content, there is very little unhydrated material left after 7 days storage to give substantial increases in strength by continued hydration. Kuhl [7] described this hydraulic hardening of cements as a process in which "the inner, deep

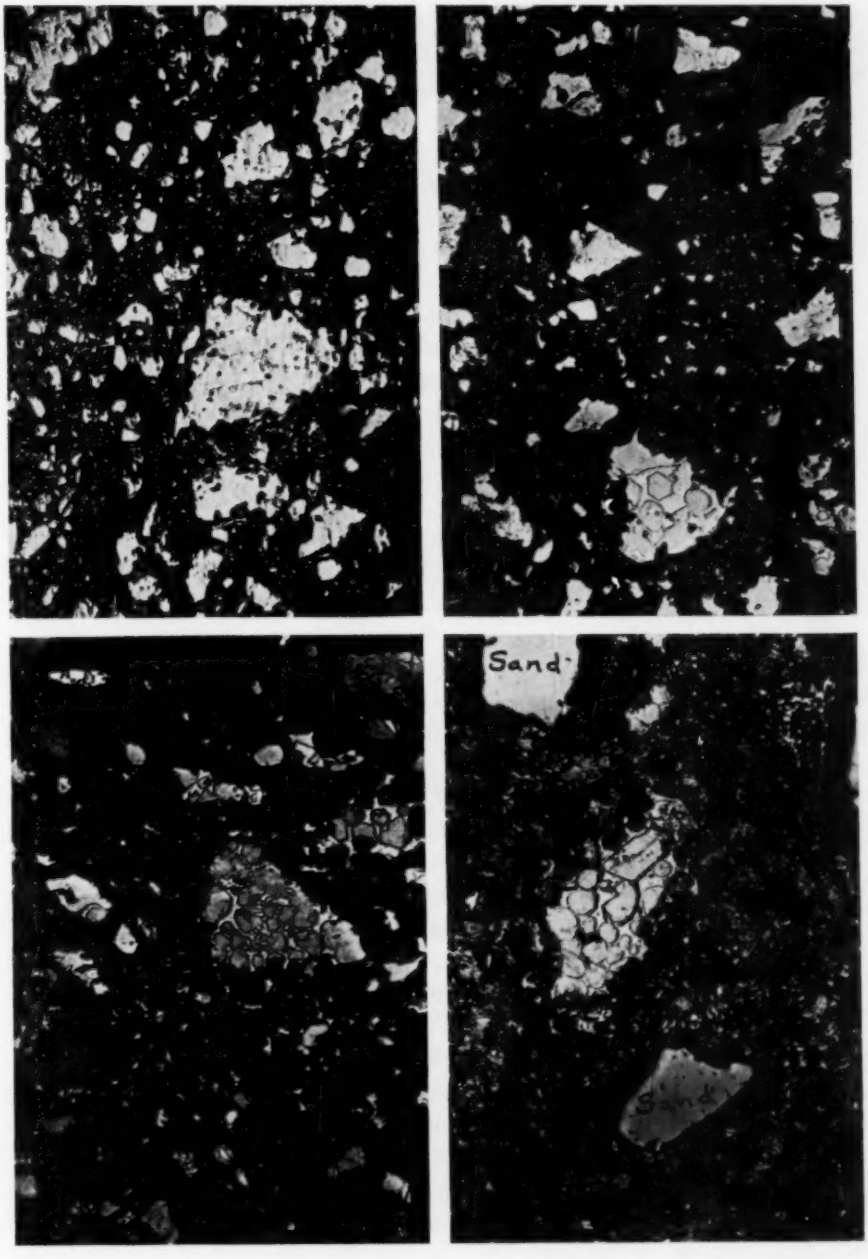

Fig. 5 to 8

Fig. 5 (upper left)—Three day hydrate, type 1 cement (220 ×). Fig. 6 (upper right)—Seven day hydrate, type 1 cement (220 ×). Fig. 7 (lower left)—Twenty-eight day hydrate, type 1 cement (220 ×). Fig. 8 (lower right)—Section from concrete pavement exposed for six years. One unhydrated cement particle, showing C_3S and C_2S crystals, at center (440 ×)

layers of cement grains are sucking off water from the hydrogel, which process is accompanied by a hydration of these layers. Gel which is on the surface of a grain envelops it more and more closely until it reaches complete water-tightness." These illustrations seem to confirm his conception of hydraulic hardening.

Fig. 5-7 show similar progressive hydration of a type 1 cement of 1800 surface area. Calculated composition of the cement showed 48 per cent C_3S, 24 per cent C_2S and 12 per cent C_3A. This cement had a 7 per cent residue on the 200 mesh screen. No attempt will be made here to estimate the rate of hydration of this cement because the size distribution curves gave no narrow classification of the particles coarser than 60 microns on which to base such calculations.

Fig. 5 shows the 3-day hydrate of type 1 cement at 220 magnification. This cement with a lower surface area than the type 3 cement of Fig. 1-4, contains a larger amount of coarse material which is plainly visible by a comparison of the photographs. Approximately 25 per cent of the cement remains unhydrated at 3 days; in the 3-day high early strength cement about 5 per cent remained (See Fig. 2). The largest unhydrated particle remaining in this type 1 cement at 3 days is about 75 microns in size.

Fig. 6 is the 7-day hydrate of type 1 cement at 220 magnification. There is a noticeable decrease in the amount of unhydrated material remaining as compared with the 3-day hydrate. Less than 20 per cent remains unhydrated at 7 days.

The largest particle in the 7-day hydrate of Fig. 6 is approximately 60 microns. In that particle at the lower center of the photograph, one well developed hexagonal crystal of C_3S is visible. Two round crystals of C_2S can also be seen in the particle, illustrating clearly the difference in crystal habit of the two silicates. C_3S generally develops with straight line edges and angles while C_2S usually assumes a rounded form. Under favorable conditions of crystallization, C_3S often develops the hexagonal form illustrated here.

Fig. 7, shows 220 magnification of the 28-day hydrate of type 1 cement. There is not much difference in the amount of unhydrated material as compared with the 7-day hydrate of Fig. 6. The amount remaining in the 28-day hydrate is estimated at about 15 per cent.

The largest particle of unhydrated cement in the 28-day hydrate is triangular in shape and 70 by 100 microns. The rounded forms of C_2S can be seen in it. C_3S can be identified in the particle to the right of and a little above the large triangular grain.

These illustrations show that as mortars age in water, there is progressive hydration of the cement particles. Primarily the smaller particles

are hydrated first of all while the larger particles gradually decrease in size but may only be completely hydrated after years of exposure.

The illustrations show that in a cement which is not too finely ground, there is still unhydrated material remaining after 28 days to continue the hydration and hardening processes by withdrawing water from the gel to seal it more firmly and to make it more and more water-tight.

Fig. 8 shows a section taken from a concrete pavement after 6 years exposure—440 magnification. A particle of unhydrated cement about 70 microns can be seen near the center of the figure. Angular crystals of C_3S and rounded C_2S can be distinguished in it. Some grains of sand are visible,—imbedded in the hydrate.

The remaining photographs of this paper, Fig. 9-25, were made from neat cement specimens stored in sealed glass vials until the time of examination. No water other than the mixing water was in contact with these specimens at any time.

Specimens 9-25 were only lightly stained with patent blue; and the photographic prints were not overdeveloped so that the contrast between hydrated material and unhydrated particles is not accentuated as it was in photographs 1-8. Accordingly in the photographs which follow, the cement hydrate appears medium grey instead of black, while the unattacked particles are light grey or white.

Fig. 9 and 10 illustrate the size distribution of particles of a type 2 cement as affected by laitance formation and bleeding. These photographs at 220 magnification were taken along a vertical plane in the cylindrical specimen. The top of the cylinder was about $\frac{1}{64}$ in. above the top of Fig. 9, which therefore represents the layer of laitance of the specimen. In Fig. 9, the size and the number of unhydrated grains increases with the depth from the top. Fig. 10 was taken about $\frac{1}{4}$ in. below Fig. 9, that is below the layer of laitance. Here the coarser unhydrated particles typical of the cement are present. These photographs show the consequences of bleeding and laitance formation. The laitance (Fig. 9) contains a higher percentage of hydrated material than is present within the body of the section as illustrated by Fig. 10.

Fig. 11 and 12 also illustrate the structure of the laitance as compared with the structure within the specimen. These two photographs were taken in planes parallel to the top of the specimen; Fig. 11 about $\frac{1}{64}$ in. below the top of the cylinder at 450 magnification; Fig. 12, 2 in. within the specimen at the same magnification. Fig. 11 therefore represents the laitance and shows only a small amount of very fine unhydrated material. Practically the entire substance of the laitance is composed of medium grey hydrates.

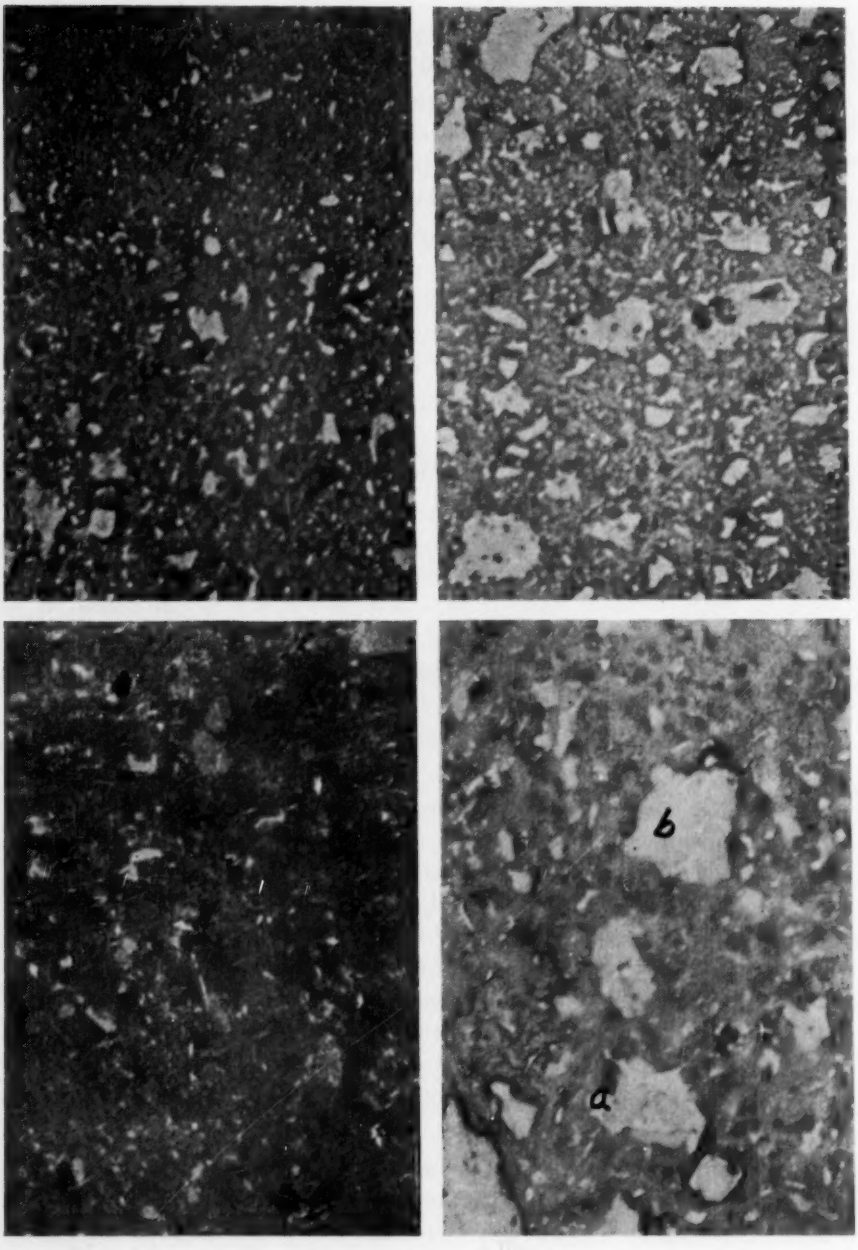

Fig. 9 to 12

Fig. 9 (upper left)—Twenty-eight day hydrate, type 2 cement. Vertical section in laitance at top of specimen (220×); Fig. 10 (upper right)—Twenty-eight day hydrate, type 2 cement. Vertical section in body of specimen ¼" below Fig. 9 (220×); Fig. 11 (lower left)—Twenty-eight day type 2 cement, horizontal section in laitance $\frac{1}{64}$" below top of specimen (450×); Fig. 12 (lower right)—Twenty-eight day hydrate type 2 cement. Horizontal section 2" below top of specimen (450×)

Fig. 12, taken 2 in. below the surface of the same specimen, shows numerous unhydrated particles of varying size imbedded in the medium grey hydrate. The section was etched in the acid etchant and outlines of C_3S and C_2S crystals can be seen in the larger particles. At the lower center, a 30 micron particle, "a," shows distinctly angular crystals of C_3S imbedded in interstitial matter. Some partly decomposed C_3S crystals are visible along the outer edge of the particle. The interstitial matter consists largely of calcium aluminates and ferrites or their combinations such as C_4AF, "glass," etc.

A little to the right and below this group of C_3S crystals, Fig. 12, there is a 10-micron crystal of round C_2S with a thin layer of interstitial matter enclosing it. Just above center of the same photograph, a group of C_2S crystals, "b," may be seen. Hydration of the latter particle has proceeded as far as the boundaries of some of the C_2S crystals, but does not seem to have attacked them noticeably.

Fig. 13 is an unetched section of the 28-day hydrate of type 2 cement at 450 magnification. There are some flaws in the section which were not entirely removed during polishing. In the center of the photograph there is a 70-micron cement particle showing round C_2S crystals imbedded in the lighter interstitial matter. A crystal of partially hydrated hexagonal C_3S also lies within the group at "a." The hydrated portion of the C_3S is darker than the unhydrated part, but still retains the angular shape of the original crystal. The hydrate was molded and confined to that shape by the shell of interstitial matter which apparently has not yet been attacked by the water. In the same group another completely hydrated crystal of C_3S, identified by its straight line edges, has merged entirely with the surrounding hydrate at one end where the confining shell of interstitial matter has been broken.

Fig. 14 is another field of the 28-day hydrate of type 2 cement at still higher magnification (900X). Along the center of this photograph there is an irregular, elongated cement particle approximately 70 microns long. It consists of medium grey C_3S and white interstitial matter. Originally when the cement was mixed with water this particle was attacked by the water from all sides to form the dark grey hydrate around it. By 28 days the reaction advanced to the irregular edge limiting the unhydrated particle in the photograph. Those portions of the C_3S crystals which have been hydrated have lost their form and have merged with other hydrates to form a compact mass around the grain.

From the photograph it is evident that the C_3S has hydrated faster than the interstitial matter which protrudes beyond those crystals and into the hydrate at the points "n."

At "s" and "s'", two semicircular outlines can be seen limiting the unhydrated particle. Those areas were probably occupied by round

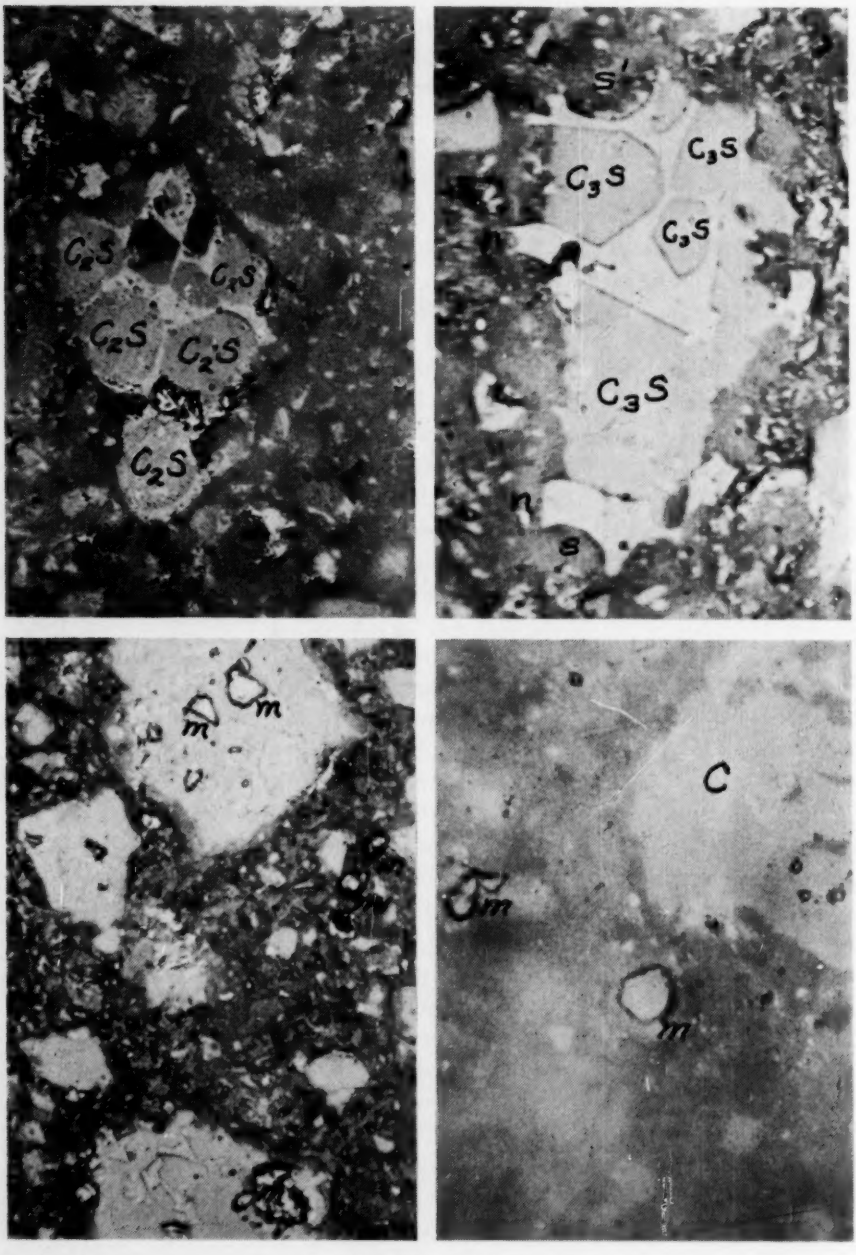

Fig. 13 to 16

Fig. 13 (upper left)—Twenty-eight day hydrate, type 2 cement. Section shows unhydrated C_2S and partially hydrated C_2S crystal "a" (450×); Fig. 14 (upper right)—Twenty-eight day hydrate, type 2 cement. Section shows partially hydrated C_2S, hydrated C_2S at "s" and "s'" and unhydrated interstitial matter at "n" (900×); Fig. 15 (lower left)—Seven day hydrate, type 1 cement, showing crystalline MgO (450×); Fig. 16 (lower right)—Twenty-eight day hydrate, type 1 cement. The MgO crystal "m" at center shows slight hydration along edges (×900)

C_2S crystals originally. At "s" hydration has been complete; but at "s'" there still remains a narrow saw-toothed skeleton of unhydrated C_2S along the curved edge.

These photographs 12, 13 and 14 show that the unhydrated material remaining in this type 2 cement at 28 days consists of C_3S, C_2S and interstitial matter, which are the three principal constituents of the clinker. These constituents undoubtedly react at different rates if all are in contact with water. But particles often are composed of large aggregates of crystals which must be decomposed from the outside before the centers can react. Accordingly no single, major constituent of cement is entirely and selectively decomposed at any age. These examinations gave no evidence of channeling of the water into the grains to hydrate the more reactive C_3S which might lie in the center of a particle. Hydration seems to proceed by the gradual reduction in the size of the particle as a function of the surface exposed to the water. Small crystals of C_2S may hydrate long before the decomposition of large C_3S crystals is complete. Fig. 14 offers definite evidence that some of the interstitial matter of type 2 cements hydrates most slowly of all. No attempt was made in these examinations to differentiate interstitial matter into its various constituents as classified by Insley [8] and others.

Fig. 15 shows a 7-day hydrate of type 1 cement at 450 magnification. The light unhydrated cement particles can be distinguished readily from the darker grey hydrates. The two largest unhydrated particles are about 100 microns in size.

This cement shows the presence of periclase magnesia, "m," which can be recognized readily by the distinctly black outlines typical of those crystals. They are present in both the hydrated and unhydrated cement. Delayed expansion of cement is often attributed to the hydration of the MgO to $Mg(OH)_2$, which reaction is accompanied by an increase in volume, setting up stresses in the cement.

Fig. 16 shows an unetched section of a 28-day hydrate of this same type 1 cement at 900 magnification. An unhydrated cement particle, "C", containing some very small crystals of MgO, lies in the upper right corner of the photograph; it can be recognized by its lighter color and irregular outline. Other crystals of MgO, at "m," are imbedded in the darker hydrate. One of these crystals of MgO in the center of the photograph shows the typical hexagonal form of periclase. It is about 7 microns in size. Close inspection of the edge of this crystal shows some pitting probably due to slight hydration. As it continues to hydrate there will be an increase in volume; it is doubtful however if a single crystal of this size would set up stresses sufficient to disrupt the cement.

All of the discussions so far have been concerned chiefly with the relation of hydrated to unhydrated material in set cements. Fig. 17-24,

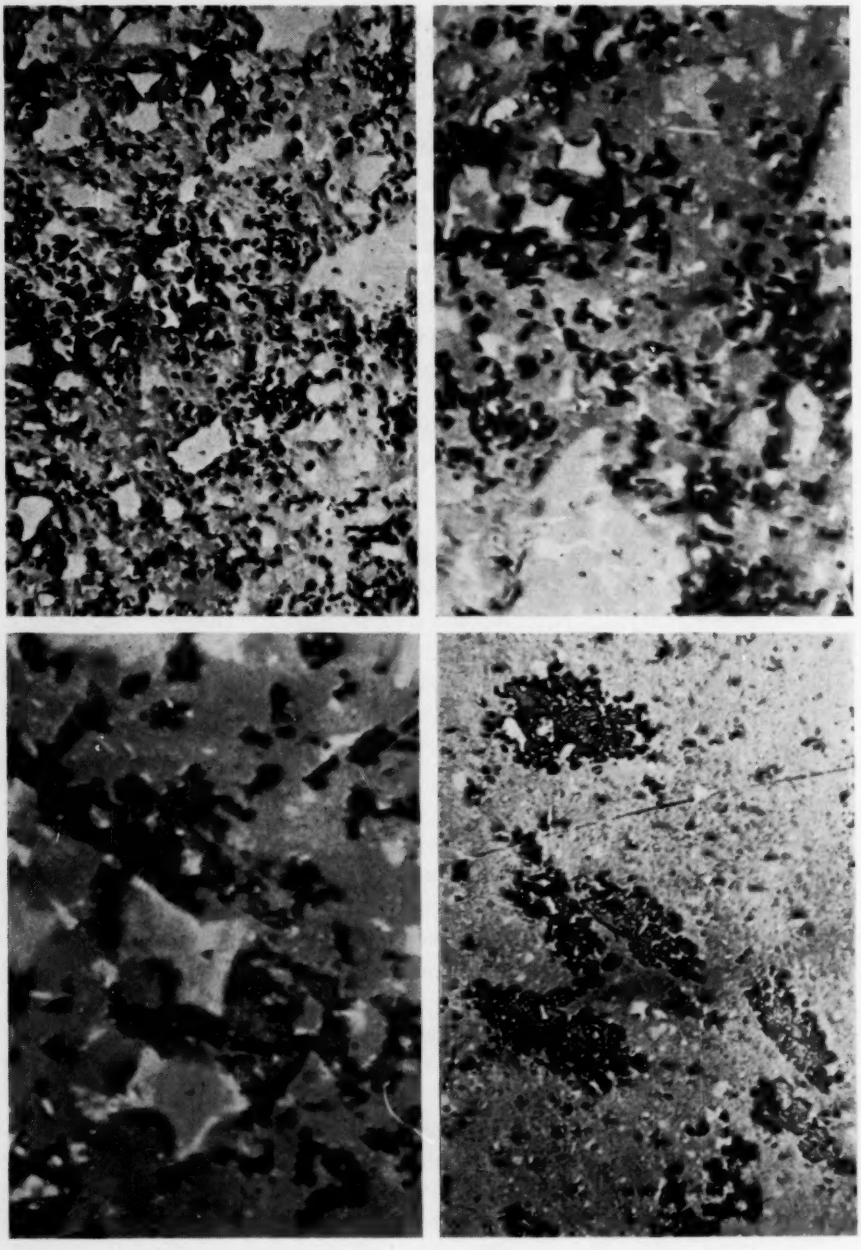

Fig. 17 to 20

Fig. 17 (upper left)—Twenty-eight day hydrate, type 2 cement, etched to show $Ca(OH)_2$ (dark crystals) (220 ×), Fig. 18 (upper right)—Twenty-eight day hydrate, type 2 cement, showing $Ca(OH)_2$ at higher magnification (440 ×), Fig. 19 (lower left)—Twenty-eight day hydrate, type 2 cement showing $Ca(OH)_2$ at highest magnification (900 ×), Fig. 20 (lower right)—Twenty-eight day hydrate, type 2 cement, section in laitance etched to show $Ca(OH)_2$ crystals (220 ×)

are more particularly designed to illustrate some of the structural details of the hydrate itself.

Fig. 17 is a 220 magnification of a type 2 cement hydrate stored in a glass vial for 28 days. The cement contained 40 per cent C_3S, 31 per cent C_2S, 6 per cent C_3A and 13 per cent C_4AF. The section was etched in water at 15 C. for one second. The water etched the $Ca(OH)_2$ of the set cement, which appears as the finely crystalline, black constituent distributed throughout the section. The lightest areas of the photograph are the unhydrated cement particles. The medium grey background consists of hydrated silicates, aluminates, ferrites, etc.

Fig. 17 shows that a considerable amount of $Ca(OH)_2$ is liberated when cement hydrates. In this section the amount of $Ca(OH)_2$ is estimated at about 10-15 per cent. The crystals are present throughout the mass of hydrogel and often are present in groups or agglomerates. Concentrations of $Ca(OH)_2$ are more plainly evident in Fig. 18,—a portion of the same field as Fig. 17, but at a magnification of 440X. Fig. 19 shows the $Ca(OH)_2$ of the same section at the highest magnification (900X).

Fig. 17-19, illustrating the distribution of the $Ca(OH)_2$ in cement hydrates, were made of sections about 2 in. below the top of the specimen. It was interesting to compare those sections with similar sections made in the laitance.

Fig. 20 shows a section of the laitance etched in water at 15 C. The $Ca(OH)_2$ crystals appear as the large dark crystals (magnification—220X). Compare the size and distribution of the $Ca(OH)_2$ of the laitance (Fig. 20) with similar crystals in the body of the cement at the same magnification. (Fig. 17). The contrast is striking. These photographs show clearly that structurally the laitance is different from the body of the cement.

Fig. 21 shows the $Ca(OH)_2$ of the laitance at 440 magnification and Fig. 22 the same crystals at 900 magnification. For comparative sizes of the $Ca(OH)_2$ within the body of the cement, compare Fig. 21 with Fig. 18, and Fig. 22 with Fig. 19.

Fig. 23 shows an unetched section of a 28-day hydrate of type 2 cement about one-eighth inch below the top of the specimen and parallel to it. At the lower edge of the photograph can be seen part of an unhydrated cement particle with round C_2S crystals visible. The medium gray background represents the cement hydrate. In this hydrate two formations of interest are marked "X." These formations did not absorb the blue dye as readily as the remainder of the hydrate. On etching in water they did not have the characteristics of $Ca(OH)_2$. Their outlines suggested a hexagonal form; possibly they may be tricalcium aluminate hydrate although no confirming evidence can be given at this time.

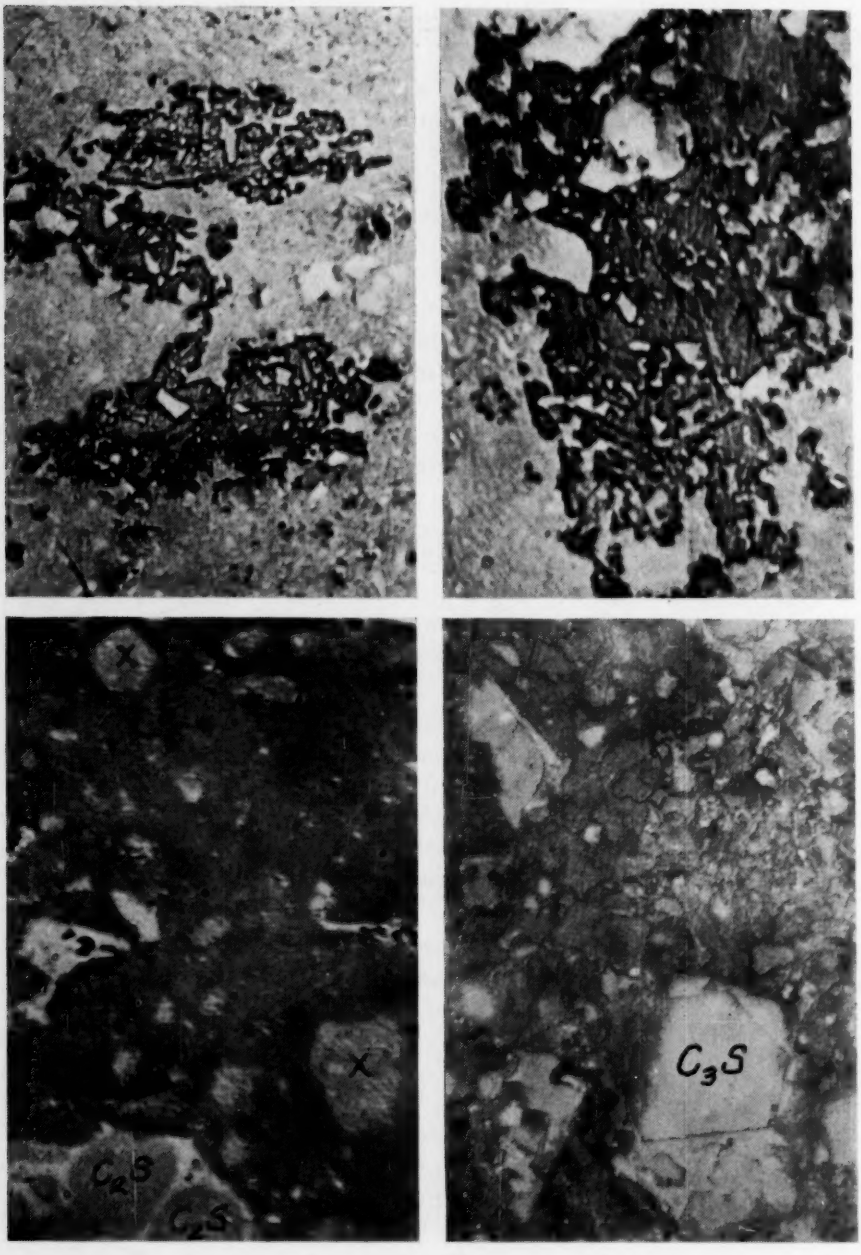

Fig. 21 to 24

Fig. 21 (upper left)—Twenty-eight day hydrate, type 2 cement, showing $Ca(OH)_2$ crystals of laitance at higher magnification (440×). Fig. 22 (upper right)—Twenty-eight day hydrate, type 2 cement, showing $Ca(OH)_2$ crystals of laitance at highest magnification (900×). Fig. 23 (lower left)—Twenty-eight day hydrate, type 2 cement. Areas "x" are unknown hydrates, possibly C_3A hydrate (900×). Fig. 24 (lower right)—Seven day hydrate, type 1 cement, etched to show structural details of the darker grey hydrates (900×).

Fig. 24 is a photograph of a 7-day hydrate of type 1 cement at 900 magnification. The section was etched for 5 seconds in a 25 C. solution containing one drop concentrated HCl in 10 cc alcohol. Particularly interesting is the large hexagonal crystal of C_3S at the lower right, which has reacted with water along its lower edge and upper left angles. The major portion of the section consists of darker grey hydrates with an extremely complicated but interesting structure. A close examination of the photograph shows that the cement hydrogel is not a formless mass but has an intricate structure. But before the various constituents of the hydrate can be identified, suitable means must be found for the differentiation of silicates, aluminates, ferrites, etc.

SUMMARY

The first photographs of this paper Fig. 1-7, show the gradual decrease in size and amount of cement particles as neat cement specimens age. In finely ground cements, surface area about 2600, very little unhydrated material remains at 7 days to continue the hydration processes; the amount remaining was less than 1 per cent in the specimens examined. In cements of surface area 1800, about 15 per cent of unhydrated cement remains after 28 days of storage. Accordingly in cements not too finely ground, there is unhydrated material left at 28 days to continue the hardening process by the withdrawal of water from the gel to render it more and more water-tight as ageing continues.

An examination of the unhydrated material remaining after 28 days storage, shows the presence of C_3S, C_2S, and interstitial matter (C_4AF, "glass," etc.). There is no microscopic evidence of channeling of water into the interior of cement particles to selectively hydrate any single, major constituent. Hydration seems to proceed by the gradual reduction in the size of the particles as a function of the surface exposed.

C_3S undoubtedly reacts with water more rapidly than C_2S, if equal surfaces are exposed. There is evidence from the photographs of this paper, that some of the interstitial matter of type 2 cements reacts with water more slowly than either of the two silicates.

A number of photographs are included to illustrate the difference in particle size distribution of the laitance as compared with similar distribution in the main body of the cement. The laitance contains the finer fractions of the cement which float to the top of the specimen; this has generally been accepted as the characteristic of the laitance. In addition the laitance contains $Ca(OH)_2$ in very large crystalline forms as compared with similar hydroxide crystals within the body of the specimen.

In the type 2 cement examined in some detail in this report, the amount of $Ca(OH)_2$ liberated at 28 days is estimated at about 10-15 per cent.

The amount would undoubtedly vary with the composition of the cement, the amount of mixing water, etc.

This paper was designed to show primarily that metallographic methods can be used successfully for studies of the structure of hydrated cements. The conclusions drawn from the data presented here are necessarily limited, because the method has had a very limited application and development in the field of cement hydrates. This metallographic method is not only applicable for the examination of neat specimens but can be used almost as readily for examining mortars and concretes. In that field its application should prove of unlimited value in problems regarding the nature of the disintegration of concretes in salt or alkali waters, the disintegration of mortars under repeated freezing and thawing, the effect of alkalies on aggregates and other problems relating to the durability of concrete.

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Discussion of this paper should reach the ACI Secretary in triplicate by April 1, 1943, for publication in the JOURNAL for June, 1943

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Discussion of a paper by L. T. Brownmiller:

The Microscopic Structure of Hydrated Portland Cement*

By E. P. REXFORD and AUTHOR

By E. P. REXFORD†

This paper is an excellent contribution to the subject of cement hydration. In this writer's opinion the author has succeeded admirably in accomplishing the primary purpose of his paper by demonstrating with an excellent selection of photomicrographs the feasibility of adapting metallographic methods to the study of hydration and the structure of cement hydrate.

The writer has successfully used the same metallographic method in the study and comparison of rates of hydration of neat portland-cement specimens, and in the study of the structure of hydrated cement in mortars and concretes. Mr. Brownmiller's observations, which included neat specimens at ages up to 28 days, were fully substantiated by such studies. Examinations by the writer of similar materials at various ages up to six months have shown that while the size and the number of undecomposed cement particles decreased markedly with age, representatives of all of the cement compounds which were present in the initial stages were also present at the end of a six-month period. Similar phenomena have also been noted in 26-year old concrete. No indication of completed hydration of any one compound was ever observed although incipient selectivity was often visible around the periphery of a cement particle. Invariably, hydration appeared to have started at the surface of a particle and to have progressed inward toward the center at a comparatively uniform over-all rate which affected all of the cement compounds of that particle more or less simultaneously. The amount of selectivity which has been observed was similar to that mentioned by Mr. Brownmiller and appeared to be of minor importance where the hydration of the entire grain was concerned.

*ACI JOURNAL, Jan. 1943; *Proceedings* V. 39, p. 193.
†Petrographic microscopist, Central Concrete Laboratory, U. S. War Department, Mt. Vernon, N. Y.

In determining the rate of hydration by means of the microscope, the author made certain assumptions. While these assumptions appeared to be very logical, it is regrettable that a series of measurements and photomicrographs could not have been made on certain individual cement particles during the hydration period to substantiate the assumptions.

Mr. Brownmiller stated that his estimates of the amounts of unhydrated cement shown in his photomicrographs were entirely visual and therefore were only approximations. The writer wishes to emphasize the inadvisability of placing too much reliance in estimates of the amount of unhydrated cement particles observed in one or two specimens as being representative of the cement as a whole. Some time ago the writer made careful micrometric analyses of neat cement specimens at various ages with a Wentworth micrometer stage to determine the relative rates of hydration of plain portland cement and treated portland cement. Numerous separate specimens from the same cement batch were made for each of a series of ages. All were treated in exactly the same manner from beginning to end. They were all ground to the same height, polished, and etched in the same manner, yet micrometric counts of one four-day old specimen showed 6 per cent of undecomposed cement by volume, while a companion specimen showed 15 per cent at the same age. Comparable 60-day specimens have also shown similar differences. Such wide divergence is not usual, but the fact that such ranges may be encountered emphasizes the necessity of analyzing as large a number of specimens as possible if accuracy is to be obtained by the metallographic method.

The author has suggested the use of tin oxide as an abrasive for the final polishing of neat specimens. This material has not been tried here, but very successful polishes have been obtained with a fine grade of jewelers' rouge on chamois. The rouge imparts a light pink color to the cement-hydrate ground mass while the unhydrated particles stand out clearly without the necessity of dyeing the specimens for photographic purposes as was required with the tin oxide. Where the pink color is a disadvantage a fine grade of white emery on chamois has been found to be very satisfactory.

Extensive microscope studies of the development of calcium hydroxide in set cement have also been made. In many cases, examinations of this material were made by means of very thin, etched, polished thin sections where the birefringence of the calcium hydroxide under fully or partially crossed nicols made it easily distinguishable from its surrounding medium. The advantage of the etched polished thin section over the normal type of polished section lies in the fact that both reflected and plain or polarized transmitted light may be used. The use of the different types of light makes it possible to study simultaneously other features of the specimen

as well as the calcium hydroxide without changing the position of the thin section. This method, however, has the disadvantage of requiring carefully made polished thin sections which necessitate long periods of hand lapping, not only to acquire a sufficiently smooth surface, but to obtain the extreme thinness necessary in thin sections of cement hydrate. The results, once obtained, however, are usually worth the effort if time is not the major factor.

AUTHOR'S CLOSURE

Mr. Rexford has brought out an important point in his discussion, which probably should have been given greater emphasis in the original paper, referring specifically to his comment 4. As he points out, variations in structure occur even when the experimental conditions are controlled most carefully.

The selection of photographs in the original paper was made after a study of quite a number of preparations; and those photographs were chosen as the more typical examples for the conditions described. Nevertheless, they represent one man's endeavor within a restricted time, and therefore, are subject to limitations in a field of such wide scope where discrepancies occur which cannot easily be ascribed to any specific cause. Hence it is desirable that innumerable examinations and measurements be made by a large number of laboratories before generalizations are made too freely.

It is gratifying to know that the Central Concrete Laboratory, and other laboratories also, have been using this metallographic method successfully for studying hydrated cement and concrete structures with results which agree in many details. Early publication of the results from these laboratories is urged in order that those interested may benefit from the experience of others.

Mr. Rexford's short description of his technique and of his use of polished thin sections is a timely contribution and should be noted by those who are using the method. A more detailed description of his procedures and results would be most welcome as a further guide for the development of this metallographic method.

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Job Problems and Practice

"JPP" is a means toward realizing more fully than in a limited number of longer JOURNAL contributions, the mutuality of ACI Membership effort to do a better concrete job. In JPP many Members may participate in few pages. So, if you have a question, ask it. If an answer is of likely general interest, it will be briefed here (with authorship credit unless the contributor prefers not). But don't wait for a question. If you know of a concrete problem solved—in field, laboratory, factory, or office—or if you are moved to constructive comment or criticism, obey the impulse; jot it down for JPP. Remember these pages are for informal and sometimes tentative fragments—not the "copper-riveted" conclusiveness of formal treatises. "Answers" to questions do not carry ACI authority; they represent the efforts of Members to add their bits to the sum of ACI Member knowledge of concrete "know-how."

From a "Charming Old Port Town" in the Southwestern Pacific

By C. C. EVERHART

[Mr. Everhart, of Moville, Iowa, an ACI member since 1925, writes Oct. 20 from "A Southwestern Pacific Port," to pay his dues. He was a "Pearl Harbor Volunteer" (Dec. 11, 1941) and is with the 14th Naval Construction Battalion—better known as the "Sea Bees."—EDITOR]

We are temporarily tied up in a charming old port town of (censored) origin. We have had a lot of idle time aboard ship and I wanted to spend some of this time making a study of the concrete masonry in this town.

The retaining wall design (industrial) is of the counterfort type very light in section and apparently functioning well. The designers have reduced their yardage of concrete in these walls to an absolute minimum, far less than American practice.

I have noticed many examples of architectural concrete all well constructed of thin sections, some even as thin as standard two inch lumber. This work shows great skill in workmanship and good modern architectural design. These thin sections show no signs of rust or scaling.

The concrete homes here are beautiful and well designed and the workmanship is excellent. The design follows the best as laid down by

the famous (censored) modern designers (of Europe). Their colored plaster and concrete compares well with the very finest U. S. work. The dock (piling and slabs) walls show serious deteriorations due to too light a structural design and by the harbor's salt water action on the concrete.

I examined recently a magnificent example of stone wall construction. The section (foundation) of this wall along the beach had scoured out some and this disclosed a probable concrete foundation. The inscription plate on this masonry was dated 1885.

A Method of Mix Control for Light Weight Aggregates (39-120)*

By H. G. COLLINS†

Comment received since this subject was discussed in the November JOURNAL indicates a need for clarifying the statement "pre-wetting of the aggregate is not essential to controlling batch weights." Let us disregard for the moment any purpose in pre-wetting the aggregate except that of controlling batch quantities. The C.C.F. (constant cubic foot) method makes it possible to determine quickly the weight of a unit volume of the aggregate in a compacted condition which will be uniform regardless of the initial moisture content. The effective quantity of aggregate in this volume will be practically the same, although the moisture content may vary from sample to sample. This variation in moisture content will be reflected in the weight per C.C.F. Because of its speed, the test can be made frequently and the batch weights can be corrected as often as a significant change is found. Therefore, pre-wetting is not essential for this purpose.

As an example of the application of the method, let us suppose that, by means of laboratory or field trials, the following mix was decided on and the weight per C.C.F. of the light weight aggregate as used was as follows:

Quantities per Cubic Yard	
Cement	9.2 sacks
Coarse Aggregate	540 lbs.
Fine Aggregate	670 lbs.
Weight per C. C. F.	
Coarse Aggregate	45.0 lbs.
Fine Aggregate	56.5 lbs.

From this, it follows that the number of C.C.F. of coarse aggregate required will be $540 \div 45 = 12.0$, and that of fine aggregate will be $670 \div 56.5 = 11.85$.

In the job operation, however, we get the following results on a determination of weight per C.C.F.:

*See ACI JOURNAL, Nov. 1942, p. 139.
†Lehigh Portland Cement Co., Allentown, Pa.

Coarse Aggregate	48.5 lbs.
Fine Aggregate	61.0 lbs.

The corrected batch weights will be:

Coarse Aggregate	= 12 x 48.5	= 582 lbs.
Fine aggregate	= 11.85 x 61.0	= 723 lbs.

Subsequent variations in the weight of the aggregate may be corrected in the same manner with the object of maintaining a reasonably uniform effective volume of aggregate.

In this discussion, we have considered only the batching of aggregate by weight. It should be obvious that, if batching is done by volume, the only real change is elimination of the step of weighing the loose aggregate placed in the C.C.F. measure. One would simply use a ratio between the loose volume and the compacted volume.

Concrete in Water Containing Structures

By F. L. FITZPATRICK*

The subject of permeability of concrete has received a good deal of attention from research institutes, but much still remains to be explored. Unfortunately, from the point of view of the practical concrete man, much of the research does not bring within its scope certain major factors.

Of these, one of the most important is the initial state of the sample—as to moisture content—at time of testing.

Everyone experienced in making and testing concrete pipes knows that there is an enormous difference in permeability between a perfectly dry concrete and one that has been immersed in water for, say, 24 hours prior to testing.

Particularly is this so in the case of reinforced samples. This is not surprising when one considers that drying shrinkage must produce a state of compression in the reinforcement and consequent tension in the concrete. Conversely, concrete, when saturated, swells and produces a state of mild pre-stressing, the concrete then being placed in compression.

It is no doubt due to this pre-stressing factor, that reinforced concrete pipes used under hydrostatic pressure always give a higher resistance (in the pipe lines) against rupture, than similar pipes do, when tested in the factory testing machine. Failure by longitudinal rupture occurs when the hydrostatic pressure causes such tension in the pipe wall that the latter cannot sustain it.

The writer has seen some remarkable examples associated with this in a reinforced concrete pipeline which had been for some years in service

*Roela Limited, concrete engineers, Melbourne, Australia.

under a hydrostatic pressure of 50 or 60 psi., carrying a water corrosive by reason of extreme purity (coupled with small amounts of CO_2).

Serious deterioration of the concrete occurred throughout the pipeline, and individual failures occurred at intervals. These failures were usually cases where the concrete—when sufficiently deteriorated—allowed the contained water to burst away the wall, blowing a small hole therein, and were not longitudinal ruptures. Nevertheless, the majority of the pipes continued to sustain the pressure, although when samples were removed and allowed to dry, the concrete was soft and crumbly.

Initially, each pipe (6″ diameter, 1″ thick wall) was designed and made for a test pressure of 150 psi. and the pipes would fail by longitudinal cracking in factory test, at 180/200 psi. The concrete was then first class.

Samples removed from the main after some years' service, and with the concrete in a palpably deteriorated condition, were kept damp until tested to destruction. They then sustained test pressures ranging from 250 to 300 psi., i.e. a much higher pressure than they would withstand prior to undergoing any corrosion.

Failure (by longitudinal rupture) of a concrete pipe under hydrostatic test occurs when the combined stress in the concrete of the pipe wall reaches the ultimate strength of the concrete.

In the case of the pipes just mentioned, the ultimate strength of the concrete had deteriorated, yet the pipes gave higher test values.

Our theory is that during the years in service, the concrete swelled continuously, throwing a gradually increasing tension on the reinforcement, whilst some "creep" in the concrete also took place.

When the pressure was renewed, the pipe was in a "prestressed" state, with a substantial compression in the concrete, hence the high ultimate strength of a pipe of the deteriorated concrete.

Future research into permeability of concrete might with advantage, we think, be directed also to the effect on permeability of tension stress in the concrete.

In practice, there is usually tension in the concrete of water retaining structures such as pipes, tanks, etc. Tests of concrete free of stress probably have little relevance to the permeability of concrete when under tension.

Dealing with the matter of the progressive and continuous shrinkage of concrete—stored dry—and the reverse for wet stored samples, it seems clear that in course of time a tendency to considerable tension stresses in concrete must be set up where one face of a concrete wall is in contact with water, and the other exposed to sun and wind drying. The movements are considerable, and though partly offset by "creep," must account for many cracks developing in thick structures of this sort after long periods.

As a matter of interest to pipe maker members of the Institute, our practice here in Australia in designing reinforced concrete pipes for hydrostatic pressures (they are extensively used for pressures up to about 300 ft. working head), is: 1. Calculate the concrete tension (combined stress) due to the test pressure, taking $n = 6$; 2. Provide such area of steel that tension in steel at test pressure (considered for this purpose as steel taking the whole tension), does not exceed 16,000 psi.; 3. Provide shell thickness so that combined stress (tension in concrete) does not exceed 350 psi. at test pressure.

Actually we regard the tension stress in concrete as the important factor—contrary to the normal principles of reinforced concrete design.

Variations in the Weight of Mortar Pots Due to Humidity*

By R. F. ADAMS†

In making low head permeability tests of mortar pots,* there was a lack of smoothness in the permeability curves after the permeability rates had become approximately constant. This lack of smoothness was attributed to the variations in the humidity in the laboratory. In establishing this fact definitely, five of the empty mortar pots were weighed weekly on a Troemner scale for a period of over 2½ years. This period was considerably longer than originally planned, as a positive gain in weight was noted during the first year. Merely from curiosity the observations were continued through three winters. These pots were made from a 1:4 sand mortar, cured 7 days in the moist room and had been sitting in the laboratory empty for about 1½ years before the test was started. The average and the maximum and minimum weight change of the five pots is shown in Fig. 1.

In the fall, winter and early spring when the humidity in the laboratory is low, the weight of the pots is at a minimum. In the spring and summer when the humidity is generally high the weight increases. During the past year the humidity in the laboratory varied from a low of 13 per cent in the winter to a high of 85 per cent in the summer. The weight change does not follow a smooth curve but jumps around due to the variations in humidity from damp to dry weather. The weight of the pots tends to increase over a long period of time due probably to increasing hydration of the cement as shown by the minimum weight from one winter to the next. The pots were 1½ years old when the weighing was started and the weight has increased steadily in the 2½ years they have been under observation. This has also been noticed on concrete stored under constant temperature and humidity conditions. The concrete will dry out

*"Low Head Permeability Tests of Mortar Pots," Pearson and Adams, ACI JOURNAL, Feb. 1939, *Proc.* V. 35, p. 285.

†Lehigh Portland Cement Co., Allentown, Pa.

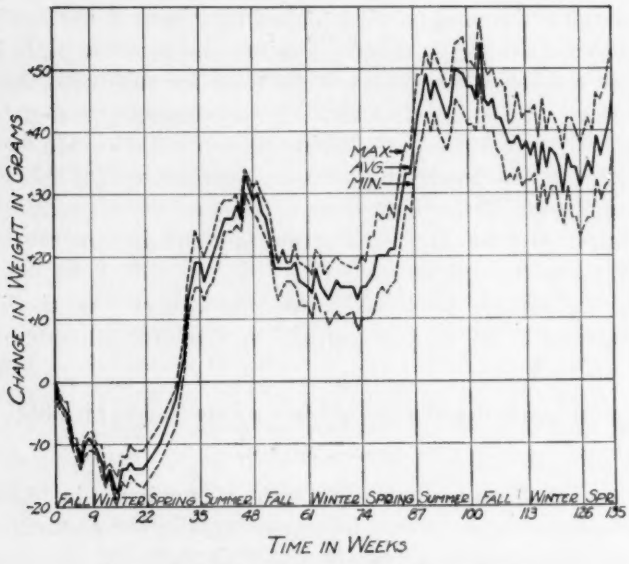

Fig. 1—Variations in the weight of mortar pots

to a minimum weight, then slowly increase in weight as the cement hydrates and takes up moisture. The maximum weight change over a period of 2 years is seen from Fig. 1 to be about 70 grams. This amounts to about 1 per cent by weight of the mortar pot.

These weight changes, while small, are of considerable importance in laboratory tests where small variations have to be accounted for, and they explain the variations in the smoothness of the curves in the paper previously referred to.

Concrete for Railway Bridge Decks, with Rails Directly Attached

Q—I should like to see a discussion of the ability of concrete to perform satisfactorily on railway bridge decks to which rails are directly attached.

A—Bridge decks with rails on cast steel chairs attached directly to the concrete are demonstrating that good concrete successfully meets this service. The climatic exposure on most of these decks is severe. Many of these decks, for example those between Toronto and Montreal, carry a large volume of traffic including a heavy tonnage of freight and frequent high-speed passenger trains. This concrete was obtained by design and control methods using sound aggregates, portland cement of both normal and high early types, with a water-cement ratio suited to the anticipated service.

Exposure on bridge decks of this type is probably not as severe as with many ballast deck bridges. With the all-concrete decks, water readily drains to the ends or drainage outlets and any water, snow, or ice that falls on the deck remains for comparatively short periods. Ballast decks, on the other hand, seldom have adequate drainage outlets and when the ballast becomes foul the water is held for long periods in the ballast trough. In winter the drains freeze upward from below and remain plugged all winter long. Water trapped on the deck then seeks other outlets such as joints, porous spots through the concrete and between layers of concrete such as the joint between the curbs and main slab which have not been properly bonded. It appears therefore that the quality of concrete to withstand the exposures on ballast decks must, if anything, be better than for solid decks.

Some of the criticism directed at the ballastless deck design seems to be based on a background of experience with rails on top of walls in engine pits. With engine pits the rails are so close to the inside edge of the walls that details must be chosen that will not aggravate the tendency to break down the inside edge. Rail chairs must not have their inside edge embedded because corrosion often breaks down the corner. Anchor bolts when set too near the edge or improperly embedded, also corrode and break out spalls. The concrete at the top of such walls should, therefore, be of excellent quality because of the moisture, oil, jacking loads and general mechanical abuse it receives.

In the case of solid deck bridges the rail chair and anchor bolts are so far in from the edge of the slab that lateral support is not a worry. The rail fastening details in use today seem to be well suited to mainline bridge deck service. The details are simple, rugged, and readily installed. They may be heavier than necessary. It should be pointed out in considering these details that should the unexpected happen and the anchorages need renewal, it is a comparatively simple matter to drill new holes in the concrete and reseat the chairs in a new location only an inch or two away from the former.

A most interesting recent installation of rail on pit walls is that at the Chicago shops of the Chicago & North Western Railway.* The rail was clipped directly to the pit walls without being set on rail chairs.

Much concrete has been made of a quality that will not successfully meet the service and exposure to which it is subjected. Too few engineers, both railway and others, do not seem to distinguish between the formance of *various qualities* of concrete. Concrete to meet any severe exposure may readily be produced with normal cement and sound aggregates. Waterproofing to protect concrete is best secured in the design and control during placing of an appropriate concrete.

*See "Concrete for Railways," V. 6, No. 4, Portland Cement Assn., Chicago.

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Some Long Time Tests of Concrete*

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and K. F. WENDT‡

SYNOPSIS

This paper reports results of strength and expansion tests on three series of concrete specimens** begun in 1910, in 1923, and in 1937. More than 2500 concrete cylinders and a still larger number of mortar briquets and cylinders were made for these test programs. Most of the specimens have been cured outdoors, some have been cured under water and others in the laboratory. Variables in types of cement, types of coarse aggregate, consistency, and methods of placement are included in these experiments.

TEST PROGRAMS

At the University of Wisconsin there are in progress three series of long time tests on the strength of concrete: Series A, specimens made in 1910; Series B, specimens made in 1923, and Series C, specimens made in 1937. Table 1 shows the number of compression cylinders, types of cement and aggregate, proportions, water-cement ratios, consistencies, and methods of placing and curing in the different series of tests. As indicated at the bottom of the table, similar mixing procedures were used in making the test specimens. Consistency measurements were not in vogue at the time of making specimens of Series A, but the records of batch weights and moisture contents of the aggregates were sufficient to provide means for estimating the slumps recorded in the table. Sufficient specimens were made to furnish 5 cylinders of a kind for each test period. In Series B and C, expansion tests are being run on three prisms of a given mix. It is planned to continue testing specimens of Series A and C for 50 years and specimens of Series B for a century.

*Received by the Institute Jan. 11, 1943.
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**See "Long Time Tests of Concrete," M. O. Withey, ACI JOURNAL, Feb. 1931; *Proceedings* V. 27, p. 547; *The Wisconsin Engineer*, Feb. 1915, Feb. 1918, Nov. 1920, and Feb. 1936.

TABLE 1—TEST PROGRAMS

Series	No. and Type of Spec.	Cement	Aggregate		*Mix, by Vol.	Method of		W/C Ratio		Slump, in.	Flow, %
			Fine	Coarse		‡Placement	Curing	Gal/Sk.	by Vol.		
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
A (1910)	450 6 x 18-in. Cyl.	Atlas (Hannibal, Mo.)	Janesville Sand	Lower Magnesian Limestone	1:2:4	Hand	Out doors In water In cellar	7.1	0.94	7 est.	
					1:3:6	Hand	Out doors In water In cellar	10.2	1.36	7 est.	
B (1923)	600 6 x 12-in. Cyl.	Medusa (3M)		Janesville Gravel	1:2:4	Hand	Out doors	5.7	0.76	3 est.	165
				Lannon Dolomite				6.1	0.81		
				Redgranite Granite				5.8	0.77		
		Lehigh (4M)		Janesville Gravel				5.7	0.76		
				Lannon Dolomite				6.1	0.81		
				Redgranite Granite				5.8	0.77		
	36 4x6x24-in. Prisms	Universal (5M)		Janesville Gravel				5.7	0.76		
				Lannon Dolomite				6.1	0.81		
				Redgranite Granite				5.8	0.77		
		Marquette (7M)		Janesville Gravel				5.7	0.76		
				Lannon Dolomite				6.1	0.81		
				Redgranite Granite				5.8	0.77		
	1000 6x12-in. Cyl.	(3M, 4M, 5M, 7M)		Janesville Gravel	1:1½:3			4.6	0.61		
					1:3:6			7.8	1.04		
					1:2:4			7.6	1.01	10 est.	250
	60 4x6x24-in. Prisms				1:2:4		In doors	5.7	0.76	3 est.	165
					1:2:4		In doors	7.6	1.01	10 est.	250

(Table 1 continued opposite page)

TABLE 1—TEST PROGRAMS (Cont'd)

Series	No. and Type of Spec.	Cement	Aggregate		*Mix, by Vol.	Method of		W/C Ratio		Slump, in.	Flow, %
			Fine	Coarse		‡Placement	Curing	Gal/Sk.	by Vol.		
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
C (1937)	480 6x12-in. Cyl.	Medusa Std. (3M7)	Janesville Sand	Janesville Gravel	1:1.66:3.80	Hand	Out doors	5.5	0.74	6.5	195
					1:1.66:3.80	Vibrated		4.5	0.60	0.7	120
					1:2.09:4.78	Vibrated		5.5	0.74	1.0	140
		Universal Mod. (5UM)			1:1.66:3.80	Hand		5.4	0.72	5.5	180
					1:1.66:3.80	Vibrated		4.4	0.59	0.5	120
					1:2.09:4.78	Vibrated		5.3	0.71	0.5	125
		Universal Std. (5M7)			1:1.66:3.80	Hand		5.5	0.74	6.0	185
					1:1.66:3.80	Vibrated		4.5	0.60	0.5	120
					1:2.09:4.78	Vibrated		5.5	0.74	0.8	135
	36 4x6x24-in. Prisms	Incor H.E.S. (I)			1:1.66:3.80	Hand		5.5	0.74	5.0	180
					1:1.66:3.80	Vibrated		4.5	0.60	0.5	115
					1:2.09:4.78	Vibrated		5.5	0.74	0.5	130

*All batch constituents weighed and mixed for 2 to 3 min. in a 6-cu. ft. Smith tilting mixer.

‡Care was taken to secure uniformity in the concrete as it was placed in the molds. To accomplish this end all cylinders of a given batch were filled one-third full in rotation, then filled two-thirds full, and then completely filled. Hand placed cylinders were rodded in a standard manner. For the vibrated batches 4 in. extenders were added to the molds; molds were completely filled; a 1⅛ in. diameter spud type vibrator direct connected to a 1/9 H. P. motor operating at 10,000 r.p.m. was introduced, and vibration continued until mortar was flushed to the entire surface.

TABLE 2—OXIDES AND COMPOUNDS IN CEMENTS BASED ON CHEMICAL ANALYSIS

Cement No.	Atlas 1910	3M 1923		4M 1923		5M 1923		7M 1923		3M7 1937	5M7 1937	5UM 1937	I 1937
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
Brand	Atlas	Medusa		Lehigh		Universal		Marquette		Medusa Std.	Universal Std.	Universal Modified	Incor
Specific Gravity		3.12		3.14		3.13		3.16					
Residue on No. 325 Sieve	41.4	31.9		30.3		29.6		26.4		12.8	12.5	11.0	8.2
Surface Area ($cm^2/gm.$)	1045	1100		1285.		1235		1295		1370	1780	1920	2110
Laboratory Analysis by	P.C.A.	W.H.C.	Mill	W.H.C.	Mill	W.H.C.	Mill	W.H.C.	Mill	Mill	Mill	Mill	Mill
CaO	60.58	60.50	61.92	62.02	61.7	63.23	64.07	62.18	62.11	63.55	64.55	63.35	65.85
SiO_2	22.18	20.99	22.19	21.55	21.8	20.21	21.18	22.01	22.14	21.19	20.98	21.52	20.36
Al_2O_3	6.04	5.50	4.93	6.98	7.5	6.80	6.54	6.75	6.69	5.48	6.24	4.99	6.88
Fe_2O_3	2.97	2.37	2.79	2.76	2.6	2.22	2.26	2.86	2.94	2.52	2.82	5.16	3.13
MgO	2.50	4.66	4.71	2.47	2.6	2.32	1.54	2.12	2.29	4.40	2.15	1.89	0.96
MnO							0.70				0.64	0.50	
SO_3	1.86	1.65	1.68	1.84	1.9	1.46	1.63	1.94	1.91	1.67	1.79	1.78	2.22
Insoluble Residue		0.26		0.18		0.22		0.31					
Free CaO						1.35		0.21					
Ignition Loss	2.78	4.24	1.22	2.34	1.4	3.32	1.70	2.64	1.47	0.55	0.74	0.73	1.00
Alkali Contents by Research Laboratory of the Portland Cement Association													
Na_2O	0.13	0.24		0.18		0.15	0.15	0.41					
K_2O	1.04	0.82		0.75		0.23	0.19	0.74					
Computed Compound Composition on Loss Free Basis													
$3CaO.SiO_2$	28.9	43.7	42.2	32.8	27.3	46.5	48.6	30.4	30.7	53.1	52.6	37.5	56.5
$2CaO.SiO_2$	44.4	29.7	33.0	38.6	43.2	25.1	25.2	41.4	41.6	21.5	21.0	34.0	16.2
$3CaO.Al_2O_3$	11.4	11.0	8.48	14.2	15.6	14.8	13.8	13.30	13.0	10.4	11.9	4.53	13.0
$4CaO.Al_2O_3.Fe_2O_3$	9.4	7.5	8.64	8.57	7.9	6.99	6.99	8.84	9.12	7.75	8.66	15.8	9.58
$CaSO_4$	3.3	2.92	2.89	3.20	3.20	2.57	2.82	3.37	3.32	2.87	3.08	3.06	3.79

MATERIALS

The cement used in Series A (1910) was bought from a local dealer. The supply of each brand of cement used in Series B (1923) was obtained from three different shipments from nearby dealers. For the tests of Series C (1937), 12 sacks of cement of each brand representing three to twelve bins were obtained from the manufacturer and local dealers.

In all series the supply for a given brand was thoroughly mixed to form a representative sample and was stored in metal cans until used.

The fineness data and both oxide and calculated compound compositions for the cements used in each series are given in Table 2. On account of the advances which have been made in the technique of cement analyses it was decided to secure new analyses for the cements of the 1910 and 1923 series. These new analyses were furnished by the Portland Cement Association Laboratory, the Wisconsin Highway Commission Laboratory, and the mills from which the cements were obtained. Of the mill analyses, that for 5M was made on a representative sample sent to the mill, the others are analyses which the mill laboratories judged representative of the cement shipped to the University.

The aggregates were all well graded. The local lower magnesian limestone used in the 1910 series was a soft rock which weathered poorly. Other aggregates were durable. Principal properties of the aggregates are given in Table 3.

CURING

The outside temperature at Madison, Wis., varies in such manner that a 6 by 12-in. cylinder will be frozen and thawed about 25 times each year and rarely does the temperature change more rapidly than 3 F. per hour when in the vicinity of 32 F. The relative humidity ranges from 65 to 85 per cent, averaging about 75 per cent. The average annual precipitation including snowfall is 32 in. The cylinders stored outside rested with their bottoms on level ground.

The water used in curing the submerged concrete specimens of Series A was changed weekly for the first two months, monthly for the next five years, and twice a year thereafter.

The cellar temperature for specimens of Series A ranged from 35 to 75 F., during the last 10 years it has been within 5° of 70 F. The relative humidity of the cellar varied between 50 and 75 per cent.

TESTING

Outdoor-cured concrete cylinders of Series A were allowed to become stable in the laboratory for a week prior to testing, while those of Series B and C stood only 2 days. Cylinders of Series A were tested with two

TABLE 3—PROPERTIES OF AGGREGATES

Series	Aggregate	Principal Constituents	Percentage Held on Sieve						Fineness Mod., m	Sp. Wt., lb/ft³	Sp. Gr.
			1½	¾	4	14	48	100			
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
A	Janesville Sand	About 60% quartz, 30% dolomite and 10% largely igneous material			6	28	91	97	3.0	109	2.66
B					3	31	88	98	2.9	113	2.70
C					0	31	86	98	2.8	113	
A	Lower Magnesian Limestone	A soft limestone of low resistance to wear or weather	0	36	95				7.1	87.5	2.74
B	Janesville Gravel	About half crushed material, ¾ dolomite, 1/5 quartz, 5% igneous material	0	43	100				7.3	105.0	2.67
C			0	40	100				7.2	105.0	
B	Lannon Dolomite	A hard durable dolomite	0	43	100				7.3	92.0	2.73
B	Red Granite	A hard durable granite mainly quartzite and feldspar	0	36	100				7.3	94.0	2.62

thicknesses of blotting paper on top and bottom until 1917, thereafter a thin plaster of Paris cap was used on the tops. Cylinders of Series B were initially capped with neat cement; defective caps were replaced with plaster of Paris at time of testing. Twenty-four hours before testing, cylinders of Series C were capped with Incor cement paste tempered with 2 per cent of calcium chloride.

Speeds of testing until 1917 were 0.1 in/min., thereafter 0.06 in/min.

MORTAR STRENGTHS

The influence of age and curing condition on the tensile strengths of standard Ottawa sand mortars made in 1910 are shown in Fig. 1. Marked reductions in strength after 10 years were caused by cracking of the neat briquets cured outdoors and by leaching of the 1:3 water-cured briquets. The average compressive strength of four 2-in. cubes of neat cement tested after 30 years exposure to the weather was 15,500 psi; for a like number cured in the cellar the compressive strength averaged 15,580

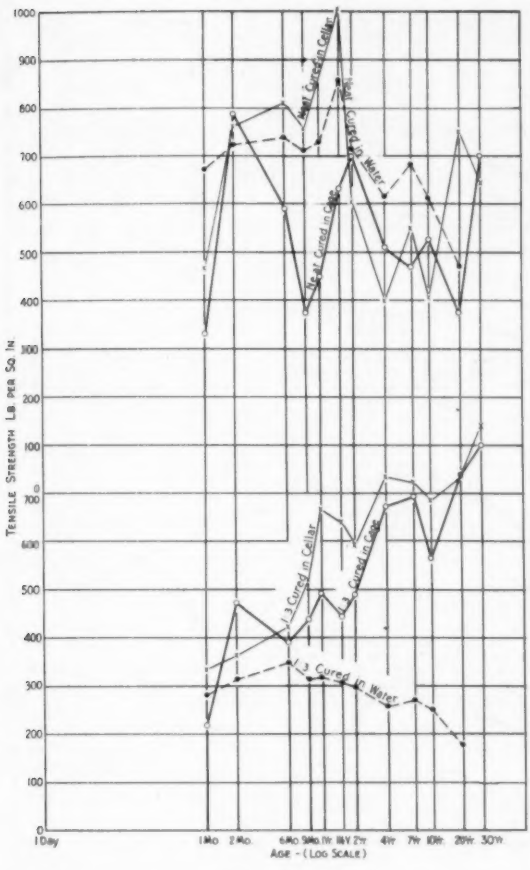


Fig. 1—Relationship between age and curing conditions on the tensile strength of Atlas cement mortar briquets of series A-1910

psi; and for four 1:3 standard sand mortar cubes cured in the cellar the 30-year compressive strength was 7,395 psi.

Fig. 2 provides strength-age relations for neat cement and mortar specimens made in 1923. Each point in the figure represents 3 tests. The reductions in tensile strength of outdoor-cured neat briquets with age due to cracking are again apparent. Also the leaching effects of water curing on the tensile strengths of the 1:3 standard sand mortars are observable. The figure shows that the rich 1:1 mortars of standard sand cured outdoors exhibited a far greater variability in tensile strength than did the lean mortars of this sand, or any of the mortars of Janesville sand. Retrogressions and variations in the mortar compression test data are much less pronounced than in tension. On the other hand few of the mortars show improvement in compressive strength between 2 and 10 years.

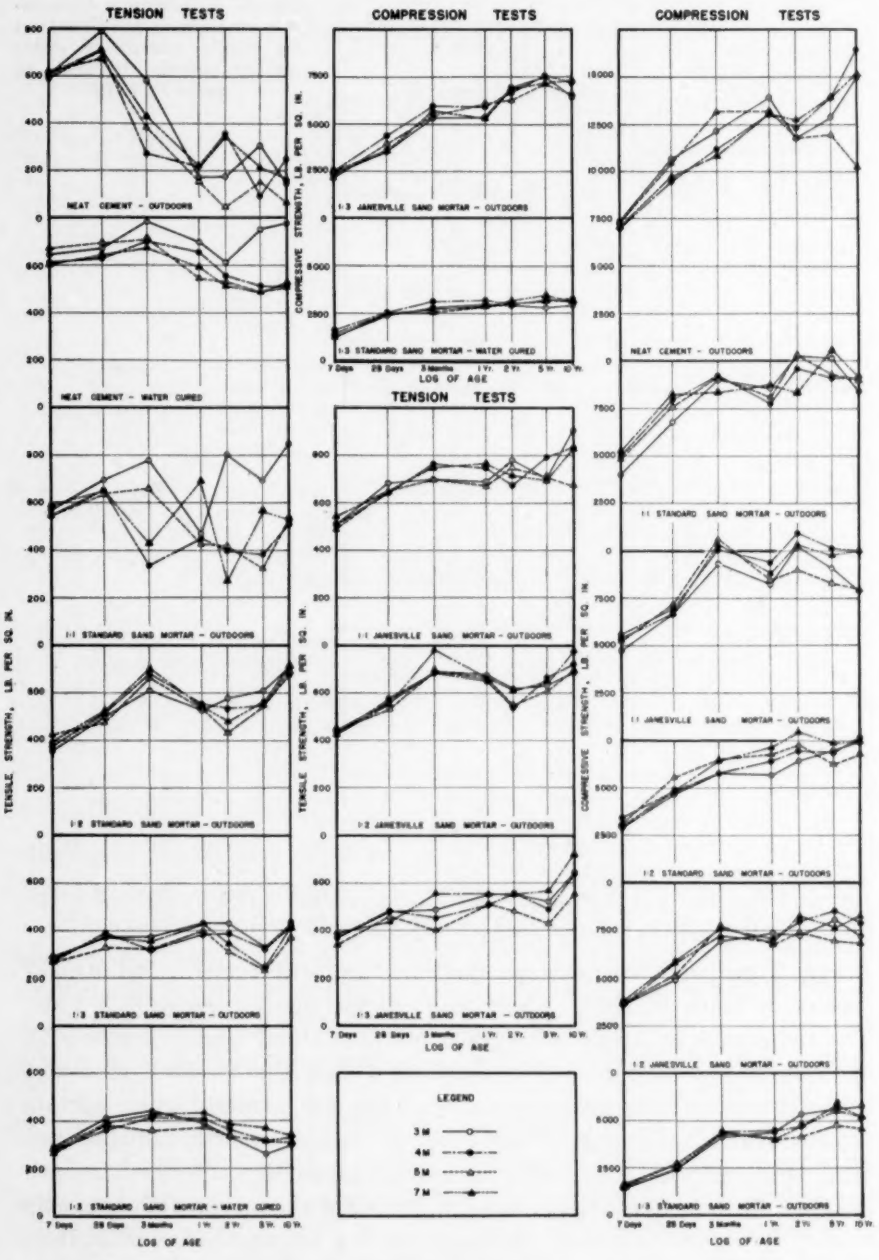


Fig. 2—Effect of age and curing condition on the tensile and compressive strengths of mortars and neat cements used in series B-1923

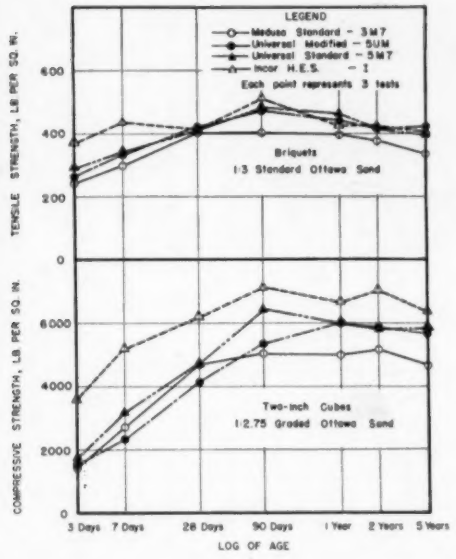


Fig. 3—Strengths of water cured Ottawa sand mortars series C-1937

Only 1:3 standard water-cured mortars were provided in Series C. Fig. 3 indicates that mortars of all of the cements of this series exhibited some retrogression in tensile strength between 3 mo. and 5 yr. In compression, retrogression is less pronounced. The mortars of the high early strength cement were markedly stronger than those of other cements at 3 and 7 days.

CONCRETE TESTS OF SERIES A

The influence of age and curing conditions on the crushing strength of the concrete made in 1910 are shown in Fig. 4 and 5. Fig. 4 shows that the strength of the cylinders cured outdoors, or in water, increased approximately with the logarithm of the age up to 20 years but at a faster rate thereafter. Fig. 5 shows the relation of strength to age plotted in uniform scales. It will be noted that the 1:2:4 mix under all three curing conditions exhibited a marked and regular increase in strength with age between 10 and 30 years, but the strength increase for the weaker 1:3:6 mix for the same period is much less marked. Examination of the outdoor cylinders of the 1:3:6 mix showed that their surfaces were slightly eroded by weathering of the paste between the sand grains. In 1937 one specimen of the seventeen remaining in the cage showed numerous cracks, Fig. 6. In 1940 the specimen was removed from the cage and both a mechanical and a chemical analysis of its contents made. These analyses showed that the cement content was normal, but there was some chert in the aggregate. It is possible that this cylinder was damaged by causes other than weathering.

Fig. 6—Appearance in 1940 of specimen No. 425 (Series A-1910) of 1:3:6 proportions.

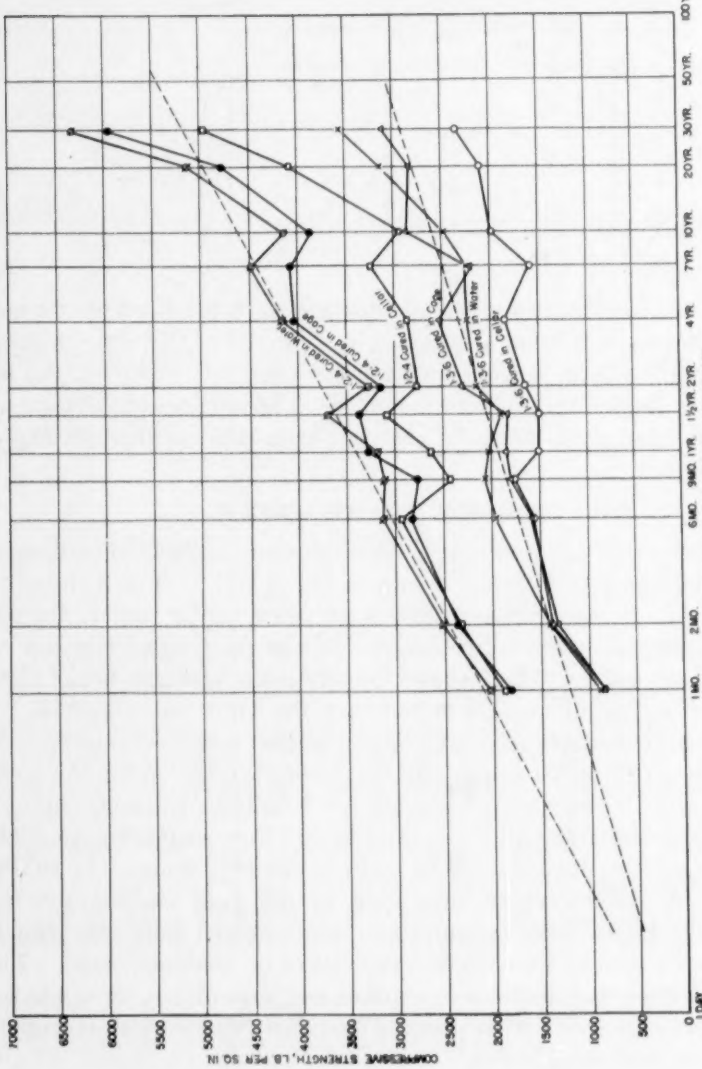


Fig. 4—Strength-age curves for concretes of series A cured in various manners

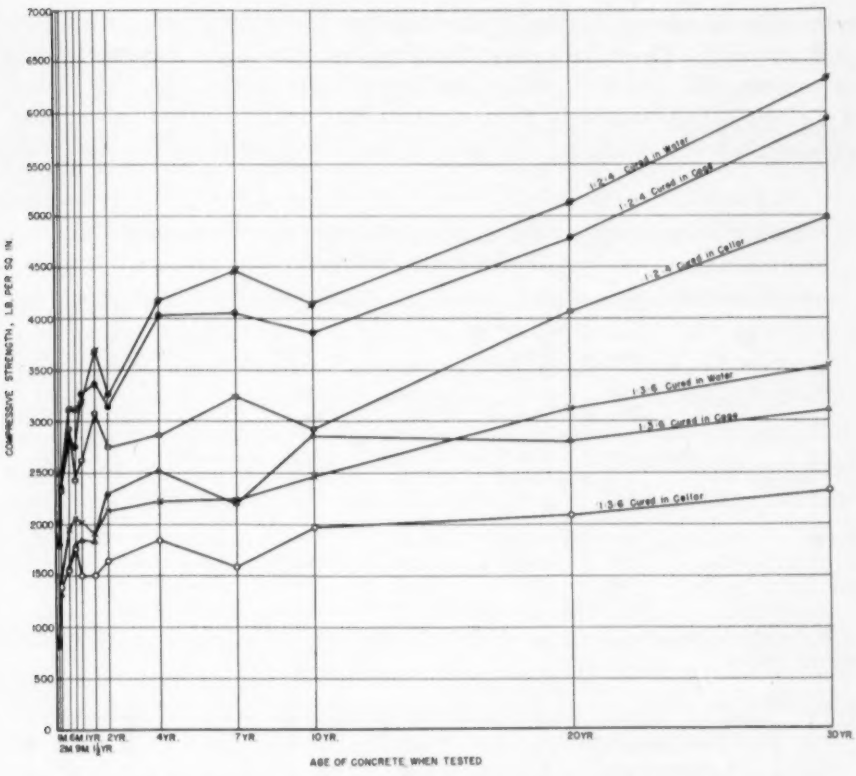


Fig. 5—Strength-age curves for concretes of series A cured in various manners

Table 4 shows the average strength, modulus of elasticity, change in weight, and Poisson's ratio values for specimens tested at 30 years. In these tests the lateral strains were measured by a Lamb's roller mirror extensometer measuring to 0.000001 in. The increase in weight of the water-cured 1:2:4 specimens with age was somewhat greater and better defined than for the 1:3:6 specimens.

CONCRETE TESTS OF SERIES B

The left half of Fig. 7 shows the strength-age relations for concretes made with three aggregates. Each plotted point represents 20 strength tests. After allowing for the small discrepancies in cement contents and in water-cement ratios, Table 5, it is evident that the aggregates used in the 1923 series were about equal in strength producing properties.

At the right in Fig. 7, strength-age curves for concretes made with the same brand of cement are shown. Each point represents 15 tests. In this portion of the figure the differences in cement contents and water-

cement ratios are uniformly dispersed in the average represented by each plotted point. These data show that the strengths of the concretes made of cements 7M and 4M, which had lower 3 $CaO.SiO_2(C_3S)$ and higher 2 $CaO.SiO_2(C_2S)$ contents than cements 3M and 5M, were somewhat greater than the strengths of concretes made with cements 3M and 5M.

TABLE 4—STRENGTH AND ELASTIC PROPERTIES OF 30-YEAR OLD CONCRETE OF SERIES A

Each strength value represents 5 tests, other values represent at least 3 tests.

Mix, by Vol.	Cured in	Average Weight, lb./cu. ft. at		Per cent Change in Weight	Average Crushing Strength, psi.	Modulus of Elasticity, E, at ¼ Ultimate, psi.	Poisson's Ratio at ¼ Ultimate
		Making	Testing				
1:2:4	Outdoors	150.1	148.5	—1.13	5955	5,130,000	0.18
w/c = 0.94	Water	147.5	150.6	+2.08	6340	5,450,000	0.19
(by Vol.)	Cellar	147.7	145.0	—1.86	4965	4,420,000	0.15
1:3:6	Outdoors	147.5	143.9	—2.45	3095	4,510,000	0.17
w/c = 1.36	Water	147.3	149.1	+1.18	3555	4,650,000	0.18
(by Vol.)	Cellar	147.8	142.1	—3.89	2330	3,720,000	0.16

The 1:3:6 specimens all exhibited slightly roughened surfaces due to weathering of surface paste.

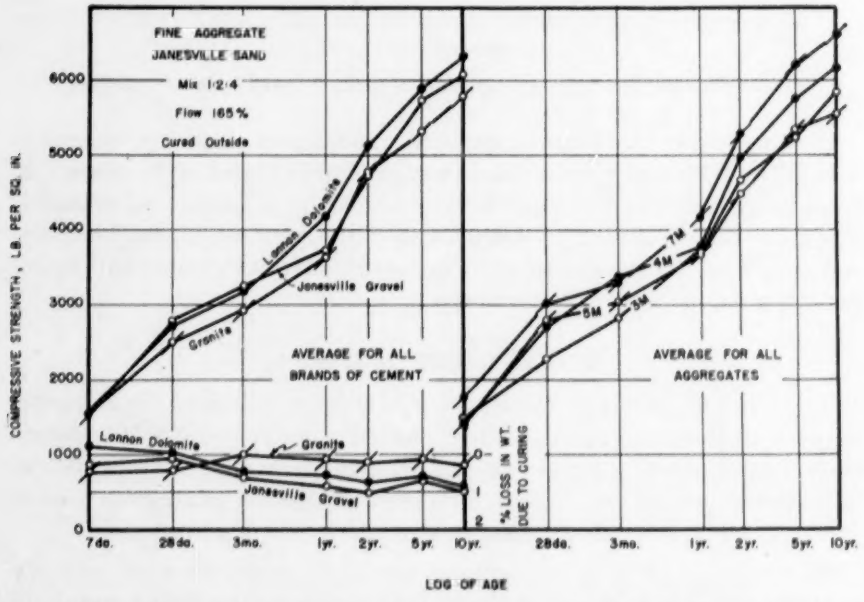


Fig. 7—Strength-age relationship for concretes of series B made of different brands of cement and different aggregates

Loss in weight and contraction measurements indicated that the granite concrete specimens exhibited less change in weight or length than did the specimens of the other aggregates.

The detrimental effects of a relatively dry atmosphere were evidenced by the tests of concretes cured 28 days in the moist room and then indoors (Table 1) at a relative humidity range between 20 and 50 per cent. At 3 months these specimens attained a crushing strength of 3000 psi which was superior to the strength of the concrete cured outdoors, but thereafter the strength-age curves for these dry-cured specimens show little increase in strength up to the 10-year period. At 10 years the indoor cured concretes had an average strength of 3400 psi, where similar concretes cured outside acquired an average strength in excess of 5000 psi. The indoor cured gravel concrete specimens with 165 per cent flow contracted about 0.04 per cent in 10 years whereas the estimated contraction of similar outdoor specimens was about 0.015 per cent.

CONCRETE TESTS OF SERIES C

Fig. 8 to 11 show the influence of age and method of placement on the strength, contraction and change in weight of concrete made with four modern cements. These specimens were all cured outdoors. For each of the cements the vibrated concrete with w/c = 0.60, by vol., gave the highest strength, the least change in weight and the lowest contraction. At 5 years the strength of the 1:1.66:3.80 vibrated mix of low slump was 17 to 32 per cent greater than the strength of the more fluid mix of like cement content placed by hand. Comparing the two mixes with w/c = 0.74 it will be observed that at most ages the vibrated mix was slightly stronger than the hand placed mix.

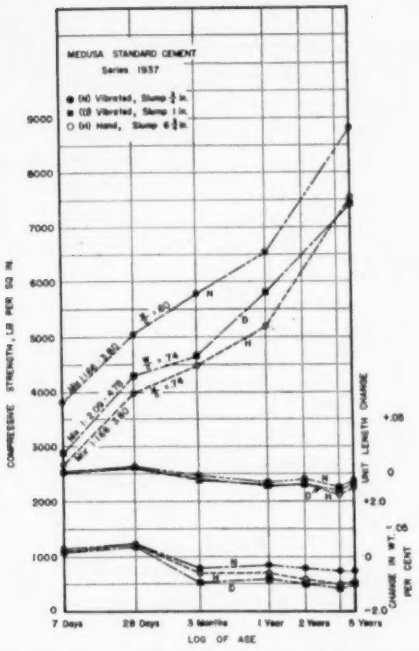
Comparing the concretes of the 1937 series made with the standard cements it is evident that the concretes made with the finer ground cement 5M7 were somewhat stronger in the early ages than those made with 3M7. At 5 years, however, the strengths were practically the same. On the other hand the contraction of concretes made with 3M7 was the least of all concretes in this series.

The strengths of the concretes made with the finely ground modified cement 5UM were surprisingly high at ages of 1 month or over, and at 5 years the highest in the program. The rates of increase in strength with age of the concretes made with this cement were the best of those tested. It will also be noted that the C_3S content, Table 2, for this cement is relatively low, the C_2S content is fairly high, and the $4\,CaO.Al_2O_3.Fe_2O_3$ is substantially greater.

TABLE 5—A COMPARISON OF THE COMPRESSIVE STRENGTHS OF CONCRETES AT DIFFERENT AGES WITH THE COMPOUND COMPOSITIONS OF THE CONSTITUENT CEMENTS

All specimens cured outdoors after moist curing one month.
Series A test results adjusted to 12-inch cylinder length for purposes of comparison ($S_{12} = 1.05\ S_{18}$).

Test Series	Coarse Aggregate	Mix Proportions		Cement Content, Sk/yd.³	Water Cement		Method of Molding	Cement	Compound Composition of Cement			Surface Area, cm²/gr.	Compressive Strength, psi. at				
		by Vol. C.S.G.	by Wt. C.S.G.		By Vol.	By Wt.			C_3S	C_2S	C_3A		1 mo.	1 yr.	5 yr.	10 yr.	30 yr.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)
A	Lower Mag. Limestone	1:2:4	1:2.18:3.50	5.75	0.94	0.63	Hand	Atlas	28.9	44.4	11.4	1045	1920	3440	4250	4050	6250
		1:3:6	1:3.27:5.25	4.01	1.36	0.90	Hand		28.9	44.4	11.4	1045	875	1920	2550	3010	3240
B	Average of Janes. Gr. Lannon Dol. Red Granite						Hand	3M	43.7	29.7	11.0	1100	2285	3735	5250	5870	
		1:2:4	1:2.4:4.47	5.15	0.76	0.51	Hand	4M	32.8	38.6	14.2	1285	2715	3775	5780	6185	
		1:2:4	1:2.4:3.92	5.52	0.81	0.54	Hand	5M	46.5	25.1	14.8	1235	2740	3690	5735	5570	
		1:2:4	1:2.4:4.00	5.41	0.77	0.51	Hand	7M	30.4	41.4	13.3	1295	3005	4165	6215	6610	
C	Janesville Gravel	1:1.66:3.80	1:2:4.25	5.68	0.74	0.49	Hand	3M7					3990	5195	7510		
		1:1.66:3.80	1:2:4.25	5.82	0.60	0.40	Vib.	3M7	53.1	21.5	10.4	1370	5030	6530	8780		
		1:2.09:4.78	1:2.51:5.34	4.73	0.74	0.49	Vib.	3M7					4285	5805	7400		
		1:1.66:3.80	1:2:4.25	5.66	0.74	0.49	Hand	5M7					4525	6015	7540		
		1:1.66:3.80	1:2:4.25	5.82	0.60	0.40	Vib.	5M7	52.6	21.0	11.9	1780	6185	8195	9015		
		1:2.09:4.78	1:2.51:5.34	4.75	0.74	0.49	Vib.	5M7					4990	6240	7820		
		1:1.66:3.80	1:2:4.25	5.72	0.72	0.48	Hand	5UM					5150	7395	7940		
		1:1.66:3.80	1:2:4.25	5.84	0.59	0.39	Vib.	5UM	37.5	34.0	4.53	1920	6265	8310	10430		
		1:2.09:4.78	1:2.41:5.34	4.76	0.71	0.47	Vib.	5UM					5130	7325	8860		
		1:1.66:3.80	1:2:4.25	5.67	0.74	0.49	Hand	I					5145	6715	7640		
		1:1.66:3.80	1:2:4.25	5.82	0.60	0.40	Vib.	I	56.5	16.2	13.0	2110	7050	8695	9995		
		1:2.09:4.78	1:2.51:5.34	4.74	0.74	0.49	Vib.	I					5480	7360	8405		



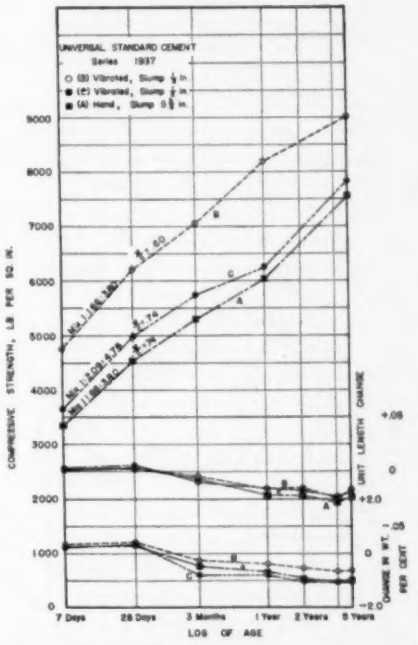


Fig. 8 (left)—Effect of mix and placement on the compressive strength, length change and loss in weight, of concrete made with Medusa standard cement

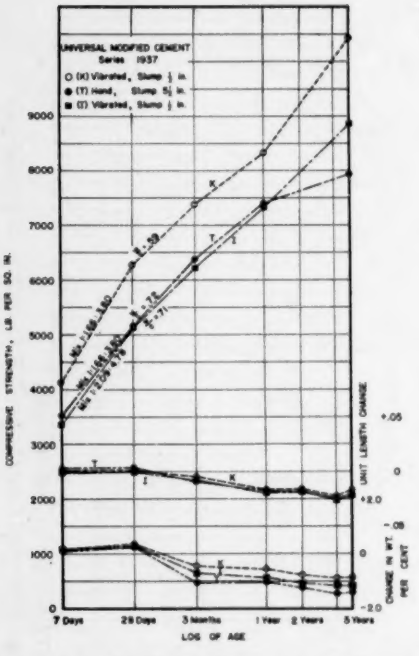
Fig. 9 (right)—Effect of mix and placement on the compressive strength, length change, and loss in weight, of concrete made with Universal Standard cement

COMPARISON OF RESULTS OF THE DIFFERENT SERIES OF TESTS

In Table 5 are data which afford some comparisons regarding the strengths of concretes made in these different test programs, also some comparisons with regard to the cements used in making the concretes. It should be noted that each strength result given for Series B represents the average of 15 tests which includes the three coarse aggregates for each cement. Fig. 12 furnishes a comparison of the strengths of concretes of the different series which were cured outdoors and tested at ages up to ten years. The curve for the 1:1.66:3.80 mix of 1937 represents the average data for concretes made with the standard cements 3M7 and 5M7. The data shown for the 1923 series are for the gravel concretes only.

The strength-age curves of Fig. 4, 7, and 8 to 12 indicate that the strengths of the concretes of all series increased roughly with the logarithm of the age up to five years. For the concretes of series B this relation held to 10 years and for series A up to 20 years.

A comparison of the strength-age curves for the 1:2:4 gravel concretes of series B, Fig. 12, with flows of 165 and 250 per cent, shows that the use of excess water causes a greater loss in strength after 5 or 10 years than



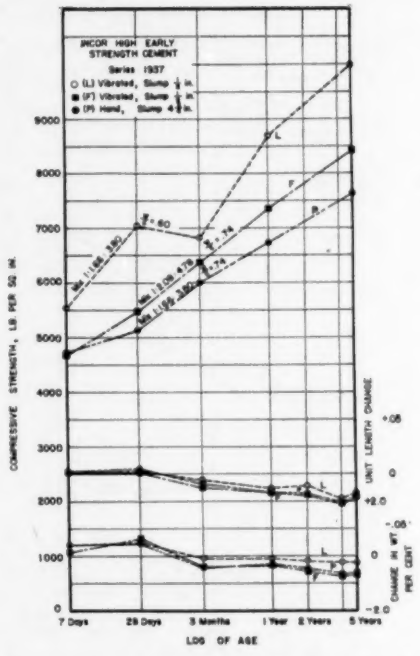


Fig. 10 (left)—Effect of mix and placement on the compressive strength, length change, and loss in weight, of concrete made with Universal modified cement

Fig. 11 (right)—Effect of mix and placement on the compressive strength, length change, and loss in weight, of concrete made wtih Incor high early strength cement

at earlier ages. The strength of this wet 1:2:4 mix is only slightly above that of the 1:3:6 mix of the same series.

Making allowance for the discrepancies in water-cement ratios and length of cylinders used in the 1910 tests it appears that the concretes of the 1923 series were slightly stronger at 10 years than those of the 1910 series. The data of Table 5 and Fig. 8 to 12, indicate very clearly that the concretes made in 1937 were much stronger at any age up to five years than those of comparable water-cement ratios made in the earlier series of tests. For example: after making allowance for the small difference in water-cement ratios, it can be seen from Fig. 12 that the 1:1.66:3.80 mix of 1937 is about fifty per cent stronger than the 165 flow, 1:2:4 mix of 1923 at an age of 7 days, and nearly 25 per cent stronger at 5 years.

Most prisms of series C exhibited considerable surface checking within one year after making, but there have been no further developments of these surface defects. Such defects were minor for the concretes made with cement 5UM and were not noted in the specimens of the earlier series.

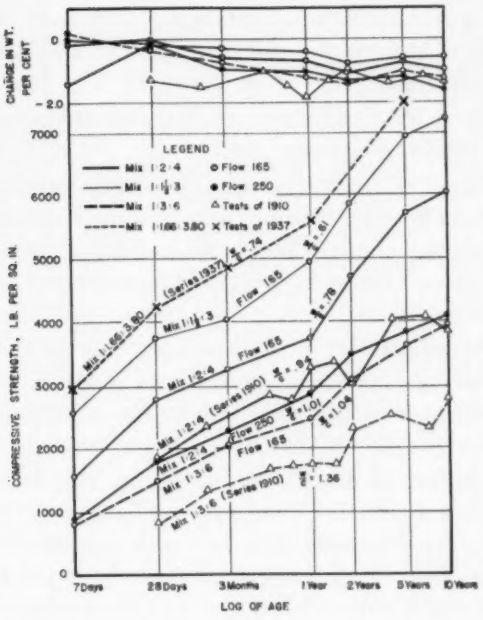


Fig. 12—A comparison of the compressive strengths of outdoor cured concretes of series A, B and C

The high ten-year strengths of the concretes made of cements 4M and 7M and the high five-year strengths of the concretes made of cement 5UM support the argument that finely-ground cements made with low proportions of tricalcium silicate and relatively high proportions of the dicalcium silicate produce concretes with high strengths at late ages.

CONCLUSIONS

1. The tensile strength of mortar suffers more from outside exposure than does the compressive strength. Neat cement and 1:1 standard sand mortars exhibited marked variability in strength during these tests.

2. The compressive strengths of concrete made with cements of the type used in 1910, 1923 and 1937 showed a good increase in strength, in accordance with the logarithm of the age, when cured unprotected on relatively dry ground in the climate of Madison. Curing indoors at lower humidities markedly reduced the rate of increase in strength after 3 months.

3. Both mortar and concrete tests of series B (1923) and the concrete tests of series C (1937) show that in each series the cements highest in dicalcium silicate gave the highest strengths at the 5 and 10 year periods.

4. The retention of excessive amounts of mixing water during the hardening of concrete resulted in a pronounced lowering of the rate of

strength increase in the concrete of 250 per cent flow. The actual difference between the strength of concrete of 165 and that of 250 per cent flow was greater as time increased.

5. The data in series B showed little difference in concrete strength which can be attributed to the type of aggregate used.

6. The contractions and losses in weight of the specimens stored indoors were about double the values for specimens of the same age cured outdoors. The water-cured concretes of series A exhibit a small increase in weight with age. The rate of increase is better defined in the richer 1:2:4 mix than in the leaner 1:3:6 mix.

7. The improvements in strength and volume constancy due to the use of an internal vibrator in placing concrete are well shown in the data of series C. Both of these increases were probably due to the reduction in water content which was made possible by the use of the vibrator.

8. Although the outdoor exposure is not the most severe, the data from the tests thus far completed show that concretes and mortars of usual proportions made of the types of cement used and good aggregates had very good weathering qualities. It is estimated that during the 30-year exposure specimens of series A suffered 750 cycles of freezing and thawing. Under these conditions the strengths of the 1:2:4 specimens increased from 1800 psi at 1 month to 5955 psi at 30 years and the strength of the 1:3:6 mix increased in nearly the same ratio.

ACKNOWLEDGMENTS

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Bomb-Resistant Air Raid Shelters*

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Member American Concrete Institute

SYNOPSIS

This paper outlines the problems involved in the structural design of two types of bomb-resistant shelters and emphasizes the inadvisability of any large program of shelter construction in the United States.

A shelter to resist the direct hit of a 1000-lb heavy-case bomb uses a roof slab 11 ft. 3 in. thick, sidewalls 6 ft thick above ground and 9 ft. thick below, and a base 5 ft. thick.

A surface shelter to resist blast and splinters, from a 1000-lb bomb exploding 25 feet away, employs 15 in. sidewalls, and an 8-in. roof.

The author closes with a plea and a suggestion for better outside walls for future buildings.

INTRODUCTION

Since it is questionable whether any structure designed for civilian protection can be "bombproof," in other words, absolutely safe against all of the effects of modern, high-explosive projectiles, "bomb-resistant" shelters will be considered. It is possible to design structures which will stop heavy armor-piercing projectiles. It is another matter to build such structures. The sections required are so massive and costly as to preclude their construction for anything except installations dictated by military necessity at strategic points, such as major control centers in the theatre of operations, or big gun emplacements guarding important harbors. And, even though such structures may be considered as bomb-proof today, they may prove to be vulnerable tomorrow. Man's genius for destruction seems to exceed his capacity for construction.

What degree of resistance shall be incorporated in shelters for civilians? Shall they be designed to resist direct hits? If so, what size and type of bomb shall be used as the criteria?

Or, shall civilian shelters be designed to resist only the blast pressures and the shell fragments from nearby explosions? If so, what size of bomb and what minimum distance from the point of explosion shall

*Received by the Institute Dec. 24, 1942.
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govern? Should the roof in this design be made just strong enough to resist the impact of a debris load? How big a load, falling from what height? Or, because of the large number of incendiaries which may be dropped, should the roof be designed to resist the direct hit of a heavy incendiary bomb?

These questions are difficult to answer and the same answers will not apply for all locations. Economic considerations, availability of materials, vulnerability of location, and public pressure are important factors which will affect any public program of shelter construction. It seems to be generally conceded, however, that it is economically impossible to build a large number of shelters to provide protection against direct hits of heavy bombs. The design illustration (Fig. 1) demonstrates this. Even the construction of shelters which will provide protection only against blast and splinters entails a tremendous financial burden, not to mention the use of labor and materials which are needed elsewhere in the prosecution of the war effort.

In view of this, is it advisable to provide any public shelters for civilian populations, shelters actually designed and built solely for the purpose of giving protection against aerial attack?

I believe that most people in the United States would say "No" at this time. On the other hand, most people in England or Italy or Germany or Russia would probably say "Yes." It depends chiefly upon whether or not you have been bombed. Undoubtedly, an air raid on any coastal community in the United States, even though on a small scale, would result in a great public outcry for better protection. We must not forget that about 50,000,000 people live within the 300 mile zones bordering our Atlantic and Pacific coasts. It is important then, that facts are available to present to the public the inadvisability of large-scale shelter construction.

SHELTERS TO RESIST DIRECT HITS

Let us assume that the suggestion has been made to provide semi-buried shelters of rectangular cross-section which will resist the direct impact and explosion of a 1000-lb. light-case or heavy-case bomb striking with a velocity of 1000 ft. per sec. What thicknesses are required for the roof slab, side walls, and base slab, if concrete with a strength of 3000 psi is to be used?

The designer soon realizes that he is faced with a problem that is entirely outside the realm of conventional problems in the design of reinforced concrete structures. A few facts are at his disposal, but beyond that, he is pioneering in a field where there is no established theory of design. Here are the available facts.

A 1000-lb. bomb equipped with a delayed-action fuse and striking with a velocity of 1000 ft. per sec. has a kinetic energy at impact of 15,500,000 ft. lbs. When the bomb hits a concrete slab, a small part of this energy is absorbed in heat and in deforming the case of the bomb.

Additional energy is used in pulverizing and displacing the concrete at the point of impact and in localized deformation. A crater is formed which may be as much as 4½ ft. deep,* providing the slab is thick enough to stop the bomb and prevent perforation. The remainder of the energy is absorbed by the vibrations of the structure.

With a delayed-action fuse the bomb then explodes. The explosive energy released may be 30 times as much as the impact energy. The evidence seems to indicate, however, that this will not add much to the crater already formed by the impact, because the explosion is open to the air.

The time required to stop the bomb in 4½ ft., assuming uniform deceleration, is .009 sec., a little less than 1/100 sec. This is a short time interval. However, the time for energy to be transmitted through concrete is much smaller. A value of 9000 ft. per sec. is commonly quoted as the velocity of energy in concrete and 17,000 ft. per sec. for steel. In view of this, an intense "punching-shear" effect must be manifest under the point of impact. This points to the advisability of vertical reinforcing throughout the slab and to the use of steel reflector plates on the under surface of the slab to prevent scabbing.

The *average force* acting to stop the bomb is 3,450,000 lb. This will also be the force acting on the slab for the same time interval. Although it is the force associated with a localized "punching-shear" action, it will not necessarily be the equivalent static force causing bending in the slab. This depends upon the period of the fundamental mode of vibration for the slab. If the period is less than .009 sec., the equivalent static force may be greater than the value given above.

Do we know how to calculate the period of vibration of a reinforced concrete slab? One can make a "stab" at it, using established formulas† for beams or plates, but in the calculations the following familiar questions arise to disturb one's equanimity.

*Data for estimating maximum penetration are extremely limited. Various theoretical formulas have been proposed. The best one is the Poncelet formula, formulated in 1829. Constants used in this formula, however, must be determined by actual firing tests. In order to provide a valid basis for determining the constants for concrete slabs, many slabs of different strengths and different thicknesses, must be subjected to many similar hits of the same type and different types of projectiles. A statistical basis is lacking. Nevertheless, some values are available, due to the fine investigation conducted by the Corps of Engineers, United States Army, at Aberdeen Proving Ground. A small part of their test data have been published. See "Report of Bomb Tests on Materials and Structures" issued by the U. S. Office of Civilian Defense.

A value of 4.3 ft. for an 1100-lb. bomb, presumably of medium case, striking reinforced concrete with an ultimate strength of 3400 psi is listed in Table 3, page 5, Bulletin No. 1, "Protective Construction," prepared by the Office of the Chief of Engineers and issued by the Office for Emergency Management.

A discussion of penetration formulas is given in Chapter V, "Aerial Bombardment Protection," Wessman & Rose, published by John Wiley & Sons.

†See "Aerial Bombardment Protection," Wessman & Rose, p. 115.

"What is the modulus of elasticity of concrete? What is the moment of inertia of the section? What is the effect of cracks on vibration characteristics? What are the edge conditions? Does the slab behave elastically for its entire length and width?"

It may be that the only reliable way to get the period of vibration of a restrained, two-way, reinforced-concrete slab is by mechanical means. For slabs of the thickness under discussion, however, the period will probably be smaller than the time interval for a penetration of 4½ ft. The equivalent static force, hence, will be greater than the value of the average force.

British authorities recommend using the average force as the equivalent static force for design purposes. It still is necessary to estimate penetration, however, in order to determine the force. And, curiously enough, if a higher strength concrete is used in order to reduce the penetration, the average force to stop the bomb is increased and a greater thickness of slab results from the usual design procedures for static loads. The procedure is not very satisfactory.

It has been suggested that dimensions of the structure may be obtained in terms of the energy loads of impact and explosion, and the energy capacities of the resisting materials and shapes. Little is known, however, about energy capacities of structural units and materials struck by high velocity loads and subjected to a non-uniform state of stress.*

When all of the available facts are studied, one is forced to conclude that for the time being, chief reliance must be placed upon the few values of penetration obtained from bombing tests and from observations of structures abroad which have been hit in air raids. Using such information as a guide, an estimate is made of the maximum penetration which might be expected in the slab which is to be designed. This depth is multiplied by some factor such as 2 or 2.5 in order to give the desired thickness. A factor of 2 is used for slabs (burster courses) poured directly on the surface of the ground and supported at all points. A factor of 2.5 is suggested for roof slabs of shelters.

The design of the shelter under discussion would proceed as follows: The total thickness of roof slab would be 2.5 times 4.5 ft., the estimated maximum penetration of the 1000-lb. heavy-case bomb. The result is 11 ft. 3 in. This is evidently a massive slab. The dead load, 1700 lb. per sq. ft., is so large that it is not necessary to worry about any ordinary live load in addition to the bomb impact.

The span for a slab of this depth can be almost any value desired, within reason. The Office of Civilian Defense recommends that the

*See "Aerial Bombardment Protection, Wessman & Rose, Chap. VII, for a discussion of designing for energy loads.

slab should be able to carry its own dead load with the usual factors of safety.*

This is not an explicit recommendation, for there are various interpretations of "usual factors of safety." At any rate, using the working stresses recommended by the 1940 Joint Committee Code, it will be found that spans of 100 ft. are quite possible but, in general, are not desirable because of the need for smaller compartments in shelters. A shelter with two 30 ft. clear spans and two levels is shown in Fig. 1.

There is even less information available to guide the selection of thicknesses for side walls and floor slab. A side wall above ground should be strong enough to resist a nearby explosion and to cause ricochet of a bomb striking it. From impulse-momentum relations, it is possible to calculate† the force necessary to cause ricochet, although there are uncertainties about the time interval of the force. An estimate of the resistance of a wall to a "punching" blow may be made, a factor of safety applied, and the thickness determined. It is a crude procedure, but the determination of an equivalent static force from vibration theory would only be an elegant example of pseudo-precision.

The sidewall above ground in Fig. 1 is 6 ft. thick. This is based upon the horizontal force of 1720 kips required in a time interval of 1/200 sec. to deflect a 1000 lb. bomb with a velocity of 1000 ft. per sec., arriving at an angle of 16 degrees with the vertical. Note that the thickness of the sidewall above ground is approximately one-half of that required for the roof slab.

The sidewall below ground must be strong enough to resist the tamped or confined explosion of a bomb penetrating into the ground outside and coming to rest against the wall. There is very little accumulated data, however, for calculating or "guessing" the required thickness. The severity of confined explosions was graphically demonstrated in the tests at Aberdeen when a 600 lb. bomb detonated below ground in contact with the outside face of a reinforced concrete shelter wall 5½ ft. thick. The wall was damaged severely. Enough concrete on the inside face "scabbed off" to form a pile of debris 3 ft. deep and 10 ft. long.‡ Using this as a basis, it is estimated that a 9 ft. thickness would be sufficient to resist a 1000 lb. light-case bomb. Note that a light-case bomb rather than a heavy-case bomb is now being considered. This is because the weight of TNT and the resulting explosion would be greater than with the heavy-case bomb. The reader will now understand why either the light-case or the heavy-case bomb was specified in the opening statement of the problem.

*See "Protective Construction" issued by U.S.O.C.D., p. 17.
†See "Aerial Bombardment Protection," Wessman & Rose, p. 266.
‡"Report of Bomb Tests on Materials and Structures," issued by U.S.O.C.D., p. 35.

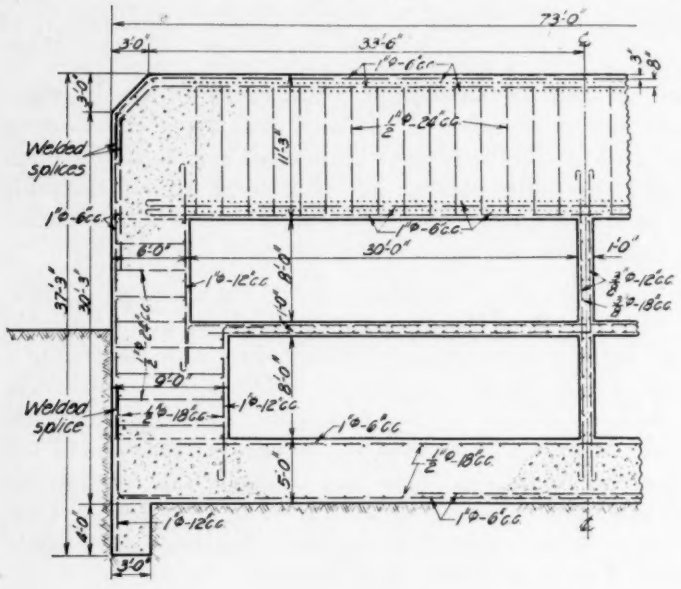

Fig. 1

The base slab must not only be strong enough to resist the distributed soil pressure from the total weight of the shelter above, but it must also resist tamped explosions. Projectiles may take a curved path through the ground and come to rest beneath the slab before exploding. If a cut-off wall is used below the side wall, however, there is practically no possibility of the bomb coming into contact with the under surface of the base slab before detonation. Hence, the thickness does not need to be as much as that part of the side wall below ground. A 5 ft. thickness is shown in Fig. 1.

It is difficult to support any specific recommendations for the reinforcing steel. The amount needed in the roof to resist the dead load stresses, considering the whole structure as a continuous frame, is negligible. The amount needed to resist bending in the roof slab caused by a bomb impact is difficult to determine for reasons already stated. The author believes that two layers of 1 inch round bars on 6 inch centers in each direction and in each face will be sufficient. It is enough to resist any bending in our two-way roof slab caused by the force of 3450 kips required to stop the 1000-lb. bomb, even though this force may not be the equivalent static force.

Note that the transverse steel is not tied to the longitudinal steel, but that four inches separates all layers. This gives added resistance to the development of planes of cleavage along mats of reinforcing which sometimes result from nearby explosions.

Vertical reinforcing in the roof slab to provide additional shear strength against bomb impact consists of ½ inch round bars on 24 inch centers in both directions anchored to both bottom and top steel. It is difficult to space these any closer and leave room for workmen during construction.

The main reinforcing in the base slab is determined from dead load stresses alone considering the whole structure as a continuous frame and considering the slab as a 1-way slab. No shear reinforcing is required in the base. All longitudinal steel in the sidewalls and base consists of ½ inch round bars on 18-in. centers.

All splices should be welded. It has been observed that bond between the concrete and steel is destroyed for considerable distances from a point of explosion and that ordinary splices are ineffective.

The reinforcing in the interior floor slab should be sufficient to resist the dead load and the live load frrom shelter occupancy. The steel in the vertical partition wall should be a minimum, say ⅜ inch round bars on 12 inch centers vertically and 18 inch centers horizontally in both faces. Note that both the floor slab and the partition wall have been made 12 in. thick in order to resist the penetration of splinters and to help confine the explosion in a single compartment in the event of a perforation by an armor-piercing projectile.

Space will not permit a discussion here of such requirements as ventilation, heating, sanitary facilities, entrances and exits, gas locks, etc., all of which will affect the detailed plan of the shelter. A brief consideration of the cross section in Fig. 1, however, is enough to demonstrate the futility of providing absolute protection for many people. Let us leave this shelter, designed to resist a 1000-lb. bomb, with the question, "What would be needed to resist an 8000-lb. bomb?"

SHELTERS TO RESIST BLAST AND SPLINTERS

The tests at Aberdeen on various types of shelters and on wall panels of different materials and different thicknesses have provided a basis for the proportioning of shelters to resist the blast pressures and metal splinters from 500 lb. or 1000 lb. bombs exploding at a point as close as 25 ft. The basis, even though supported by full-size field tests, is empirical and must not be considered as infallible. Nevertheless, we must rest our case upon these tests, supplemented to a degree by the theoretical investigations* of British engineers coupling equivalent static pressures and suctions from blast with fundamental frequencies of vibration of wall panels. A detailed discussion of design procedure will not be given here.

A cross section of a reinforced concrete surface shelter, recommended by the author to resist blast and splinters from a 1000 lb. light case bomb

*"Structural Defense," British A.R.P. Handbook No. 5, p. 11.

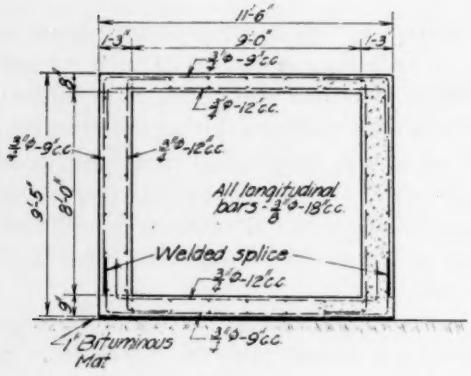

Fig. 2

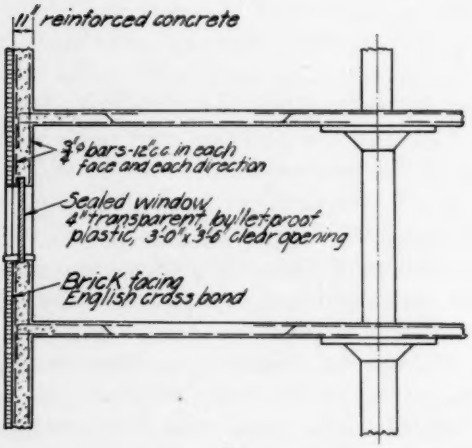
11" reinforced concrete
3/4"ø bars-12"cc in each face and each direction
Sealed window 4" transparent, bullet-proof plastic, 3'-0" x 3'-6" clear opening
Brick facing English cross bond

Fig. 3

25 ft. away, is shown in Fig. 2. Note the bituminous mat beneath the shelter, intended to permit the structure as a whole to be displaced horizontally by a nearby concussion, thus dissipating some of the energy of explosion. A shelter such as this, with a length of 54 ft. will provide considerable protection, but not safety, for 50 persons in the day time or 24 people at night. An interesting problem for the reader is to compute the cubic yards of concrete, the tons of steel, the man hours of labor required (not only for construction, but also for producing and transporting the materials) and the total cost, of providing such shelters for 25,000,000 people.

The cost is tremendous, but in these days of vast expenditure, not overwhelming. Nevertheless, it would be unwise to embark upon such a program in this country at this time. It would be much better to spend this money after the war on better cloaks for our buildings in

Courtesy Am. Inst. of Steel Const.

Fig. 4—Blast damage to an outside wall of an apartment building in London

vulnerable areas, in other words for blast and splinter-resistant outside walls for our factories, schools, office buildings, apartments, and hotels. Fig. 3 is suggestive. Outside walls, like this, might prevent some of the terrific damage illustrated in Fig. 4.

CONCLUSION

The preceding design illustrations have indicated that it is not feasible to provide protection against direct hits of heavy bombs. Even secondary protection involves a tremendous outlay of money, materials, and men. If we are bombed and a public clamor arises, the author earnestly hopes that public officials will resist pressure for a big construction program of separate air raid shelters. The possibility of intensive bombings on a large scale over here, like those in Europe, is quite remote in this war. We may have sporadic raids before the war ends, but let us take

our chances with them, making the most of the protection available in existing buildings.* Right now, it is important that all available labor and materials be used to the absolute limit on the direct war effort.

But, on the other hand, when we plan for the future, let us not close our eyes to the lessons of this war. Future wars will bring aerial attack more terrible and devastating than anything we have now witnessed. The oceans will no longer protect us. Let us encourage a wider and more intelligent use of steel, concrete, transparent plastics, and other building materials that will give, at little added cost, much greater protection against bomb splinters, blast, and fire in the buildings in which we live and sleep and work and play.

*See "Recommendations for Protection Against Aerial Bombing," issued by Metropolitan Section, American Society of Civil Engineers, Civil Engineering, Feb. 1942, p. 116.

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Tests of Composite Timber-Concrete Beams*

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SYNOPSIS

Composite beams and slabs of timber and concrete offer a very useful form of construction for heavy floors of highway bridges, piers and similar structures. The essential feature of such floors is a shear connection between the timber and concrete elements. Tests on a number of types of shear units are described in this paper, and their relative effectiveness is discussed. Working stresses are proposed and methods of applying composite construction to continuous structures are briefly outlined.

INTRODUCTION

The composite timber-concrete beam is a form of construction that has come into general use only in the last eight or ten years. In such a member, the concrete is subjected principally to compressive stress (it must be reinforced when under tensile stress), and is utilized to provide a good wearing surface under traffic and a weatherproof and fireproof cover for the timber portion of the structure. The timber, which can provide both compressive and tensile resistance, is used mainly for the latter purpose. Thus, in a simple beam the upper portion will consist of concrete and the lower portion of timber.

Floors of composite timber and concrete have been used successfully in highway bridges, hangar aprons, wharves, piers, buildings and platforms. Structures of this type will naturally be intermediate in cost between timber structures and those of reinforced concrete. They are normally used where timber is easily available and where the use does not justify the cost of reinforced concrete. Use of composite highway bridges began at about the same time at extreme parts of the United States, Oregon and Florida; examples are now found in more than a dozen states and in Canada.

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In a composite timber-concrete beam, one of the major design considerations is the provision for high horizontal shearing stresses that must exist at the junction of two materials. The neutral surface of the beam, for which the location depends upon the stiffness and size of the two composite elements, is the locus of the maximum shearing stresses, and in the usual design it will fall pretty near the junction between concrete and timber. It is evident that the natural bond of the concrete to the timber will be insufficient to resist much horizontal shear, especially for the normal case in which creosoted timber is used. Some mechanical device is thus needed to key the two parts of the beam together. Several types of metallic "shear developers" have been used for transmitting the shear across the junction of the two materials. Two groups of laboratory tests on such shear connectors were made in 1933 at other universities and the results have been published.[1] [2] The tests described here were made to secure further information on the action of such devices.

Two general types of composite timber-concrete beams are in use. In one, a concrete slab is supported on and connected with a series of timber stringers spaced some distance apart, to produce a series of parallel T-beams. This type is used in the Northwest, where large-dimensioned timbers are in common use. The other consists of a concrete slab placed on a continuous laminated deck made up of small-dimensioned timbers placed on edge. Such construction is suited to areas where the large timbers are not economically available, and has the advantage that the solid deck provides a complete formwork for the concrete slab. The tests herein apply to this second type of floor.

Obviously, this form of construction is intended principally for floors subjected to heavy loads, as in highway bridges, wharves, piers, and warehouses. It is not fireproof, although the concrete surfacing will reduce the fire hazard.

2. OUTLINE OF TESTS

The tests on 32 composite beams described here were made in the Talbot Laboratory, University of Illinois, in the period 1938 to 1942.

While the type of construction studied applies especially to floor slabs, for the sake of simplicity the tests were confined to simple beams.

An outline of the beams tested is given in Table 1. In this paper, beams of 16 types are mentioned; however, beams of Types 13 and 15 are identical in design with Types 5 and 12, respectively, differing only in age at test or manner of testing. Beams of types 0 to 11, primarily for comparison of various shear connections, were tested when the concrete had

(1) Baldock, R. H., and McCullough, C. B., "Loading Tests on a New Composite Type Short Span Highway Bridge Combining Concrete and Timber in Flexure," Oregon State Highway Commission, Technical Bul. 1.

(2) Seiler, J. F., "New Type of Composite Beam," Wood Preserving News, Vol. XI, No. 11, p. 144, November 1933.

TABLE 1—OUTLINE OF COMPOSITE BEAM TESTS

All beams were 12 ft. long, 12 in. deep, and 11½ in. wide, tested on a 10-ft. span with two concentrated loads at the one-third points of span. All timber was dense shortleaf yellow pine, structural grade, square edge and sound, S4S, with about 8 lb. of creosote oil per cubic foot. There were two companion beams of each type. Beams were cured moist 7 days after concrete was poured, then stored in air until 28 days old. All but Beams 13, 14 and 15 were tested at this age; these three types were tested at the age of 2 years, 6 months. Average concrete strength, Beams 0-11, 4300 psi.

Beam No.	Kind of Shear Connection	Spacing of Shear Connection
0	None	None
1	Triangular plates, 4 in. x 12 gage, vertical	3 rows at 4 in.
2	Triangular plates, 4 x ¼ in., vertical	3 rows at 4 in.
3	Triangular plates, 4 x ⅜ in., vertical	3 rows at 4 in.
4	Triangular plates, 4 x ¼ in., 15 deg. with vertical	3 rows at 4 in.
5	Triangular plates, 4 x ¼ in., vertical	3 rows at 4 in.
	and 60-d. spikes	4 rows at 4 in.
6	R. R. cut spikes, 6½ x ⅝ in., vertical	5 rows at 5 in.
7	R. R. cut spikes, 6½ x ⅝ in., 45 deg. incl.	5 rows at 9 in.
8	Sloped daps, 1 x 6 in.	4 rows at 8 in.
9	Sloped daps, 1 x 6 in.	4 rows at 8 in.
	and 60-d. spikes	4 rows at 4 in.
10	Lag screws, 6 x ⅝ in., vertical, 3¼ in. in wood	5 rows at 6 in.
11	Lag screws, 8 x ⅝ in., 45 deg. incl., 4¼ in. in wood	5 rows at 9 in.
12	Triangular plates, 4 in. x 12 gage, vertical	3 rows at 4 in.
	and 60-d. spikes	4 rows at 4 in.
13	Triangular plates, 4 x ¼ in., vertical	3 rows at 4 in.
	and 60-d. spikes	4 rows at 4 in.
14	Triangular plates, 4 x ⅜ in., vertical	3 rows at 4 in.
	and 60-d. spikes	4 rows at 4 in.
15	Triangular plates, 4 in. x 12 gage, vertical	3 rows at 4 in.
	and 60-d. spikes	4 rows at 4 in.

reached the age of 28 days. Types 12, 13 and 14 were made and stored in the laboratory for 2½ years before they were tested. During storage, observations were made on the shrinkage of the two materials and of any tendency toward their separation or distortion. Type 15 was used in a study of repeated loading. Two beams of each type were tested.

In designing the beams, an estimate was made of the unit value of each shear connection, and this value was used in spacing the shear units. This was done to utilize the strength of both timber and concrete and to produce a group of beams of comparable strength.

Details of the various shear connections are shown in Fig. 1. The beams were 11½ in. wide, 12 in. deep and 12 ft. long; they were tested on a 10-ft. span, with equal concentrated loads at the one-third points. The timber portion consisted of four 2 by 8 and three 2 by 6-in. pieces,

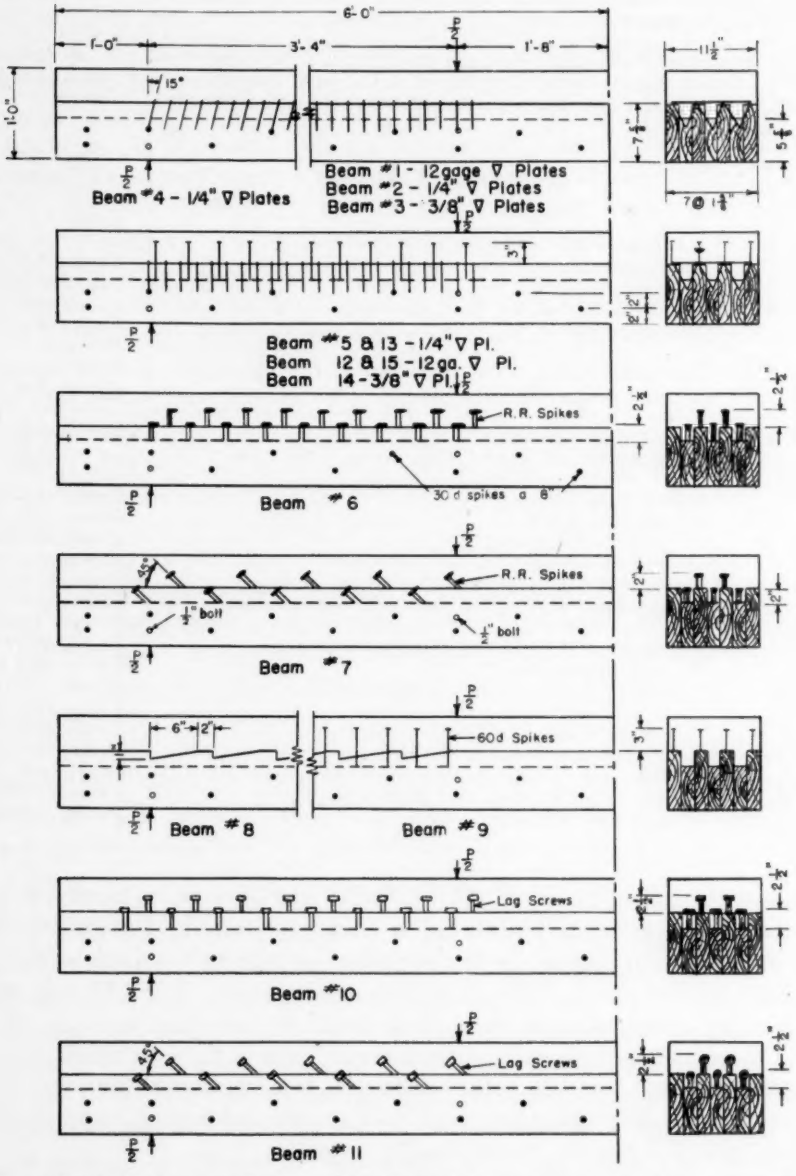


Fig. 1—Details of composite beams. For details of size and spacing of shear connections, see Table 1

well spiked and bolted together. In nine types of beam the shear connections were triangular steel plates, with side dimensions of about 4 in. In five of these types, the plates were supplemented by 60-d. spikes, which acted as ties to hold the two elements of the beams together. In type 4, the plates were inclined 15 deg. inward at the top toward midspan. Lag screws and railroad spikes used as shear units in beams of four types combined the action of a dowel and that of a tie. In general, shear units were staggered to minimize concentrations of bearing stress.

3. MATERIALS

The timber used in all of the beams was from a single shipment of dense shortleaf yellow pine, of structural grade, square edge and sound, surfaced on all sides and impregnated with about 8 lb. of creosote oil per cubic foot. The pieces, nominally 2 by 6 and 2 by 8 in., were free from any noticeable knots, checks or shakes.

For determining physical properties of the timber as received, a test was made on laminated beams consisting of four pieces of 2 by 6-in. timber, placed on edge and well spiked together, loaded at the one-third points of a 10-ft. span. The load-deflection curve from these tests was found to be a straight line up to half of the ultimate load; the computed modulus of elasticity averaged 2,050,000 psi and the modulus of rupture of the material was found to be 8170 psi.

Similar tests were made on timbers which had dried out in the laboratory for three years and thus were comparable in age with the timber used in Beams 12-15. They were intended to indicate the properties of the material used in the later group of composite beams at the time of their final testing to destruction. From these tests, the properties of the creosoted timber, after more than three years of drying in the laboratory, were as follows: modulus of elasticity, 2,250,000 psi, modulus of rupture, 7970 psi.

The concrete used was of 1:3¼:4 proportions, by weight. The water-cement ratio was 1.0, by volume, and the average slump for all batches of concrete averaged 6.5 in. The concrete materials were Medusa portland cement, Wabash River torpedo sand, and gravel of ¾ in. maximum size from the same source. The materials were mixed for two minutes in a batch mixer. The average strength of 6 by 12-in. control cylinders made from the concrete used in Beams 0-11, after 7 days of moist curing under wet burlap and 21 days storage in the air of the laboratory was 4300 psi. The cylinders made with Beams 12-14, tested when 2½ years old, had a strength of 4360 psi, while the 28-day cylinders for Beam 15 had an average strength of 5125 psi.

The metal shear connections used were apparently of a medium strength of steel, except the 60-d. wire spikes which were of high tensile strength.

Fig. 3—View of composite beam in testing machine

Fig. 2—Views of timber beams and shear developers before concreting (Beams of types 5, 7 and 9)

4. FABRICATION AND CURING OF TEST BEAMS

In fabricating the beams, the creosoted timber was assembled and spiked together and the shear connections were inserted in the timber. For the triangular plates, sawed slots, $\frac{1}{32}$-in. narrower than the plates, were provided. Similarly, pilot holes were provided for the 60-d. spikes, lag screws and railroad spikes. Daps in the timber of beam types 8 and 9 were cut by use of saw and drawknife. Views of three beams, before concreting, are shown in Fig. 2.

Side forms were attached to the timber base and concrete was placed and compacted by use of an internal vibrator. The beams were cured under wet burlap for 7 days after concreting, then were exposed to the air until they were tested.

5. TESTING PROCEDURE

The beams were tested to failure in an Olsen testing machine of 300,000-lb. capacity. Fig. 3 shows a beam in place, ready for test.

Strains were measured on eight 10-in. gage lines at midspan. These were on the top and bottom surfaces and at the sides of the beam just above and below the juncture of concrete and timber. Measurements of slip or horizontal displacement between the concrete and timber sections were taken by means of micrometer dials, located directly over the end supports, as shown in Fig. 3. A third measurement taken was the deflection at midspan, obtained by use of micrometer dials attached to the beam and bearing against wooden bars bridging between the supports.

In general, load was applied in 15 to 20 increments, with a complete set of observations at each load increment. The strains were measured by use of a Whittemore strain gage reading directly to 0.00001 in. per in. Slips and deflections were measured by dials graduated to 0.001 in.

6. PRELIMINARY TESTS

Two small groups of "push-out" tests were made to secure information needed to design the test beams.

In the first group, untreated Douglas fir timber was used. Details of the specimens are indicated in Fig. 4. The shear units studied were 4 by $\frac{3}{8}$-in. lag screws, inserted in two 2 by 4-in. surfaced wooden pieces, and then embedded in a central block of concrete. In the test, the concrete was pushed downward between the 2 by 4-in. uprights, with no external ties to hold the shearing surfaces in contact.

Fig. 4 shows load-slip curves for the three positions of lag screws used. Relative strengths and rigidities, with type B, for comparison, are:

Type	Angle of Inclination	Max. Load, lb.	Relative Max. Load	Relative Slip at Max. Load
A	45 deg.	10,160	1.60	0.48
B	90 deg.	6,350	1.00	1.00
C	135 deg.	3,475	0.55	0.51

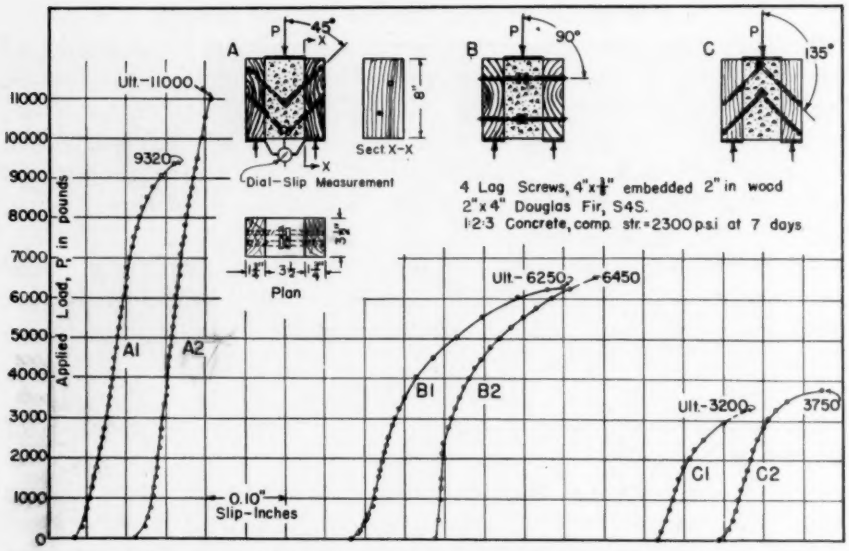


Fig. 4—Load-slip curves showing effect of inclination of lag screws as shear connections

The striking result from these tests is the high resistance of the test piece of type A, in which the lag screw acts in tension as well as shear and bending. Evidently, the lag screw acts here to pull the adjoining sliding surfaces together, thus developing some frictional resistance. The action is similar to that of an inclined stirrup in reinforced concrete.

A second group of push-out tests was made with the creosoted timber used in the beam tests. The test piece is illustrated in Fig. 5. Two were made with no shear connections and one was made with ¼-in. triangular plates. Again the concrete section was pushed downward between the timber members.

These push-out tests showed that the bond between the concrete and the finished, creosoted timber was negligible. One test developed an average bond stress of 2 psi, the other 5 psi. The third specimen, with four plate connections carried a maximum load of 24,600 lb., or an average capacity per shear connection of 6150 lb.

7. GENERAL RESULTS OF BEAM TESTS

The maximum loads carried by the various beams are given in Table 2. While a rather complete record of the individual behavior of each beam was kept throughout the tests, only a few outstanding notes are given.

It is of interest that failure in most cases occurred by tension in the timber or compression in the concrete. There was little evidence of failure of the shear units themselves, except in the case of the beams with notched daps, in which shearing failures were noted in both concrete and

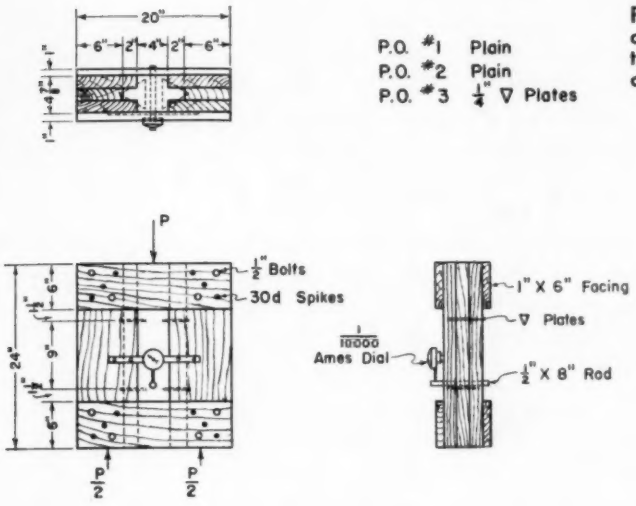


Fig. 5—Details of push-out test specimens with triangular plate shear connections

timber. In a number of cases, the concrete sheared off just above the plate shear units, and this might possibly be classified as due to faulty design of the shear plates.

The beam failures may be grouped in three general classes. The beams of type 0, with no shear units, gave the lowest strength and greatest deflection. The maximum load developed, in fact, was undoubtedly carried almost entirely by the timber portion of the beam, since at this stage the concrete had cracked extensively.

The beams with triangular plate units and spikes gave the best results as a group. They developed high loads, good integral beam action and small slips and deflections. Most of the other shear developers gave fairly satisfactory strengths and stiffnesses.

In referring to Table 2, it must be considered that since the shear connections were used at arbitrarily selected spacings, the maximum beam loads do not indicate the relative effectiveness of the different shear units. In only two beam types, those with triangular plates inclined inward at the top and those with notched daps, did the performance of the unit fall far short of the estimated value. For these, the ultimate load is only a little more than for type 0 without shear units, but the stiffness and general usefulness of these beams are far superior to those of type 0.

Beams of type 12, 13, and 14, tested after 2½ years of storage, showed no particular effect from this treatment. In general, they exhibited, during the period of storage under their own dead weight, a shortening at the top surface at midspan, amounting to about 0.0004 in. per in. at the end of the storage period. The corresponding strain at the bottom apparently varied from a small tensile strain to very nearly zero at the

TABLE 2—PRINCIPAL RESULTS OF COMPOSITE BEAM TESTS

Beams—11.5 in. wide, 12 in. deep, loaded at one-third points of 10-ft. span. Two beams of a kind.

Beam No.	Shear Connection	Age at Test	Maximum Load, Lb.			Concrete Properties		Manner of Failure
			Beam A	Beam B	Ave.	Cyl. Strength p s.i.	Mod. of Elast. Kips/ Sq. In.	
0	None	28 days	53,400	46,650	50,025	4355	4725	A, B, Tension
1	Tri. pl., 12 ga., 90 deg.	"	76,250	61,850	69,050	3905	4765	A, Tens.; B, Shear, Conc.
2	Tri. pl., ¼-in., 90 deg.	"	64,900	60,000	62,450	4505	4915	A, Tens.; B, Shear, Conc.
3	Tri. pl., ⅜-in., 90 deg.	"	64,000	70,950	67,475	4685	4290	A, Shear, Conc.; B, Comp.
4	Tri. pl., ¼-in., 105 deg.	"	47,600	53,000	50,300	4120	4260	A, B, Shear, Conc.
5	Tri. pl., ¼-in., 90 deg., 60-d. spikes	"	80,225	70,000	75,110	4055	4030	A, Comp.; B, Comp., Tens.
6	R. R. cut spikes, 90 deg.	"	77,900	75,850	76,875	4660	4720	A, Comp.; B, Tens.
7	R. R. cut spikes, 45 deg.	"	61,100	63,650	62,375	3955	4660	A, Tens.; B, Comp.
8	Sloped daps	"	51,420	52,500	51,960	4355	4680	A, Tens.; B, Shear, Conc.
9	Sloped daps, 60-d. spikes	"	64,850	61,100	62,975	4355	4655	A, B, Tens.
10	Lag screws, 6 x ⅝-in., 90 deg.	"	57,200	67,050	62,125	3955	4635	A, B, Tens.
11	Lag screws, 8 x ⅝-in., 45 deg.	"	73,400	70,230	71,815	4640	4160	A, Tens; B, Comp.
12	Tri. pl., 12 ga., 90 deg., 60-d. spikes	2½ yrs.	65,000	71,320*	68,160	4440	3910	A, B, Tens.
13	Tri. pl., ¼-in., 90 deg., 60-d. spikes	"	68,000	70,200	69,100	4560	3900	A, B, Comp.
14	Tri. pl., ⅜ in., 90 deg., 60-d. spikes	"	70,000	78,000	74,000	3910	3650	A, Comp.; B, Shear, Conc.
15†	Tri. pl., 12 ga., 90 deg., 60-d. spikes	28 days	70,300	66,300	68,300	5125	4055	A, B, Comp.

*Equivalent load for shearing stress. Actual load was 53,500 lb. applied at one one-third point only, because of damage to other end of beam.
†Repetitive loading.

end of the 2½ year period. The strains near the junction of concrete and timber were much the same for both materials and showed a shortening or contraction of 0.0001 to 0.0003 at the end of the storage period.

The strengths of these beams, when tested to failure, show very little difference from those of similar beams tested when 28 days old.

8. EFFECT OF REPEATED LOADING

Beams 15-A and 15-B were made after the other 30 beams had been tested, for the purpose of studying the effect of repetition of loading, similar to the conditions occurring under highway traffic. Except that the concrete was stronger than that used previously, these beams were duplicates of Beams 12-A and 12-B. They were tested when 28 days old.

The loading procedure was as follows: Loads were applied in increments up to 20,000 lb.; then this load was released and applied 50 times. The load was then gradually increased to 50,000 lb., and the 50 cycles of release and reloading were applied; after this, the beams were loaded gradually to failure.

The effect of the repeated loading may be judged from the recorded deflections, end slips and deformations. In general, the 50 cycles of repetition of the 20,000-lb. load produced very little increase in deflection or deformation. Since this load is approximately the safe or allowable load that would be used in the design of such beams, it follows that no great distortion or structural damage to the structure is to be expected from a considerable number of load repetitions.

The results of the repeated load cycle at the 50,000 lb. stage are also reassuring. Since this is roughly two and one-half times the allowable or working load on the structure, it is not a load condition to be expected in service. However, the distortions of the beams due to the 50-cycle repetition were not very great. The deflections at the beginning of the cycle averaged 0.50 in., and at the end they had increased to 0.59 in. With further application of load, the load-deflection curve showed a steeper slope, so that there was, in effect, some recovery of the increased deflection due to load repetition. The deformations measured at midspan showed characteristics similar to the deflections. There was little effect of the repetition of the 20,000-lb. load, but at the 50,000-lb. stage there were increases in strain of 0.0002 to 0.0003 at the extreme fibers of timber and concrete.

The end slips measured at the 50,000-lb. level were the most striking of any measurements taken. During the 50 cycles of loading there was an average increase in end slip of about 0.004 in. It may be concluded that under this load cycle there was a well-defined permanent slip and deformation in these beams, as might be expected from such a severe and

systematic overloading, but there was no serious breakdown of the member and apparently the ultimate capacity of the beam was not appreciably affected by this treatment. The limited extent of the tests must be considered in connection with these conclusions.

9. DEFLECTIONS OF TEST BEAMS

In comparing the relative stiffness of the test beams, three lines of study have been followed. First, the slopes of the load-deflection curves have been examined, since they indicate the general stiffness of the beam; second, the deflections were compared at the relatively high load of 50,000 lb., and third, a study was made of the effect of end slips upon the deflections. For this latter purpose, the theoretical deflections were computed from the measured strains at midspan, by a procedure devised by Maney,[3] and the discrepancy between computed and actual deflections was considered to be due to the slipping that took place between the two layers of the beam, and, to a slight extent, to shearing deformations.

The original data for these studies were taken from the plotted deflections, end slips and deformation, of which a typical example in Fig. 6.

Average values of the slopes of the load-deflection curves, and values of the deflections, end slips and deformations at the 50,000-lb. load are given in Table 3. Values of the load-deflection ratio, $\frac{P}{f}$, at this load represent the slope of the chord to the load-deflection curve.

From Table 3, relatively high initial values of $\frac{P}{f}$ are found for Beams 2, 3, 4, 5, 6, 7, 9, 13, 14 and 15, ranging from 103 to 119 kips per in. Beams 0 and 8 had the lowest values. Values of the $\frac{P}{f}$-ratio at the 50,000-lb. load are also quite high for Beams 2, 3, 5, 6, 12, 13, 14 and 15.

The measured deflections listed in the fourth column of Table 3 reflect much the same influences shown by the $\frac{P}{f}$ ratios. Beam 14 was definitely the stiffest of the group and Beam 0 was at the point of failure at the load shown.

The deflections computed from the measured strains average about 70 per cent of the actual deflections at the 50,000-lb. load. Thus, the end slips, which ranged from 0.005 to 0.068 in., and might seem unimportant, evidently resulted in an increase in deflection of about 40 per cent over the values to be expected had perfect integral action existed.

(3)G. A. Maney, "Relation Between Deformation and Deflection in Reinforced Concrete Beams," *Proc. A.S.T.M.*, p. 310, 1914 (see footnote, Table 3).

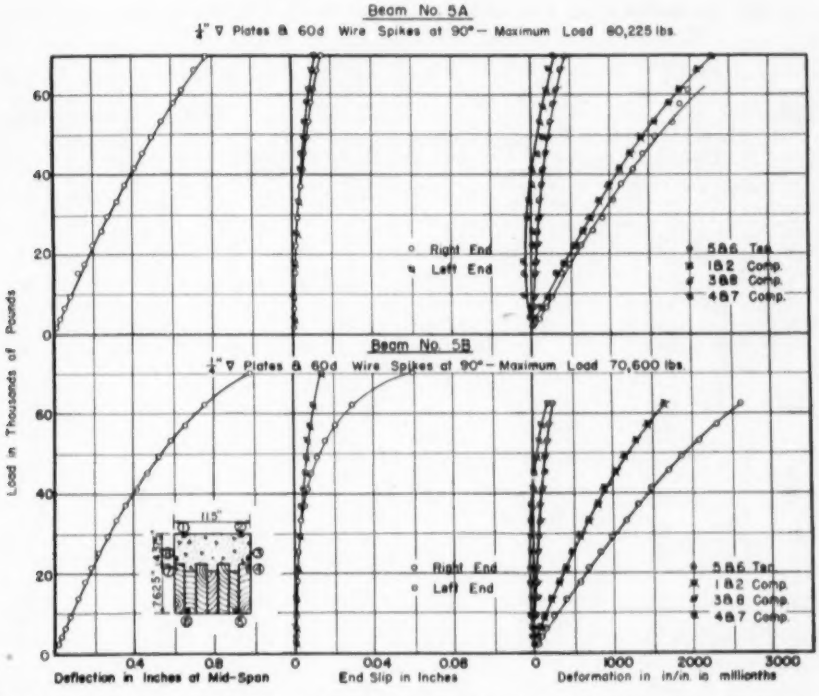

Beam No. 5A
¼" ∇ Plates @ 60d Wire Spikes at 90° — Maximum Load 80,225 lbs.
Beam No. 5B
¼" ∇ Plates @ 60d Wire Spikes at 90° — Maximum Load 70,600 lbs.
Load in Thousands of Pounds
Right End
Left End
5B6 Tar
1B2 Comp.
3B8 Comp.
4B7 Comp.
Deflection in Inches at Mid-Span
End Slip in Inches
Deformation in in/in. in millionths

Fig. 6—Typical curves showing relations between loads and deflections, end slips and deformations

10. END SLIPS IN TEST BEAMS

The sixth column of Table 3 furnishes a comparison of the average end slips of the various beams. The beams tested at the age of 2½ years are noteworthy in showing exceedingly small slips between concrete and timber at points directly above the beam supports. These slips of 0.005 to 0.008 in. are so small that they would hardly de detected by visual inspection. Of the beams tested at the age of 28 days, Beam 5 with triangular plates and spikes developed the smallest slips, followed closely by Beam 2, with ¼-in. triangular plates, and Beam 9 with daps and spikes. Beams 1 to 5, all containing triangular plates, show relatively low slips as a group, with values ranging from .008 to .022 in.; average, .015 in. All of the other beams except Beam 9 showed much larger slips, from 2 to 4 times as great as the average for the triangular plate group. It appears that the triangular plates were by far the most effective in maintaining integral action in the beams.

11. DISTRIBUTION OF STRAIN ACROSS BEAM SECTION

While the effect of slip between concrete and timber is reflected in increased deflection, the best conception of its effect upon structural

TABLE 3—AVERAGE VALUES OF BEAM DEFLECTIONS AND END SLIPS

Beam No.	Initial Slope, $\frac{P}{f}$ of Load-Deflection Curve, Kips per In.	Values at the Load of 50,000 Lb.					
		$\frac{P}{f}$, Kips per In.	Deflection, f, In.	Computed* Deflection, f, In.	Av. End Slip, In.	Max. Measured Strain, Millionths	
						ϵ_t, Timber	ϵ_c, Concrete
0	85	28	1.80†	——	.40†	——	——
1	91	88	0.57	0.39	.022	1800	1250
2	113	100	0.50	0.38	.011	1760	1200
3	113	100	0.50	0.39	.017	1680	1340
4	108	86	0.58	0.37	.019	1690	1240
5	106	96	0.52	0.39	.008	1760	1310
6	105	94	0.53	0.37	.036	1560	1310
7	107	86	0.58	0.42	.063	1840	1420
8	85	71	0.70	0.40	.068	1850	1300
9	104	91	0.55	0.40	.011	1860	1300
10	91	71	0.70	0.46	.062	2040	1600
11	94	91	0.55	0.33	.038	1530	1040
12**	98	98	0.53	0.38	.008	1700	1290
13	108	108	0.47	0.36	.005	1580	1260
14	119	119	0.42	0.36	.006	1540	1280
15‡	108	100	0.50	0.40	.011	1690	1420

*Computed by use of Maney's equation: Deflection, $f = \frac{23}{216}\frac{l^2}{d}(\epsilon_c + \epsilon_t) = 127.8\,(\epsilon_c + \epsilon_t)$

†Estimated value. Beam O-B failed at load of 46,650 lb. **Results from one beam only, 12-A.

‡Before applying 50 repetitions of the 50,000-lb. load.

action is obtained from a study of the strains measured at midspan. Fig. 6 showed load-strain curves for Beams 5-A and 5-B, and similar curves are available for the remaining thirty beams. From such curves it is possible to plot diagrams of strains at different levels, following the generally accepted principle that the variation in strain from one level to another is linear. Fig. 7 shows such diagrams for beams of three distinct types. The diagram for Beam 5-B shows the closest approach to perfect integral beam action. The linear variation from the neutral axis is almost perfect and the point of zero strain corresponds very well with the theoretical position, computed for a homogeneous beam of equivalent cross-section. This computed position coincides with the location of gage lines 4 and 7 in the timber just below its junction with the concrete. The diagram for Beam 6-B shows less perfect action. A small tension in the concrete and a considerable compression in the timber apparently exists at the junction of the two materials. This means that as a result of slip between the two elements there are actually two neutral axes, and that

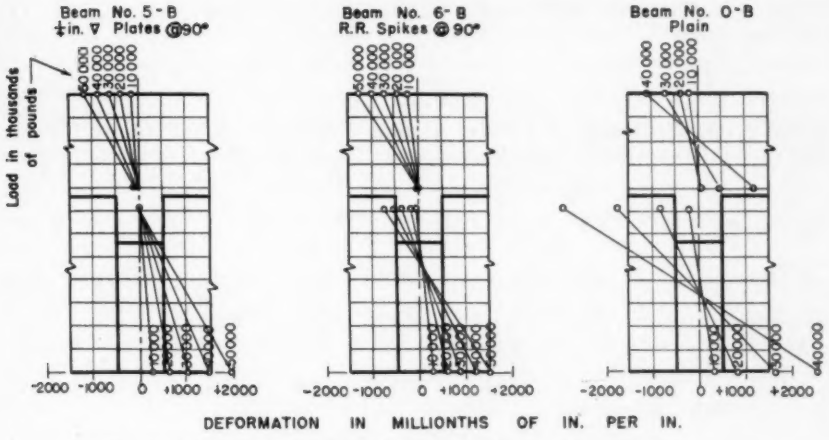


Fig. 7—Measured strains at different levels at midspan of beam

for some distance in this region the material contributes little to the flexural resistance of the beam. Finally, the diagram for Beam 0-B shows early separation of the concrete and timber layers, with each portion acting practically as an independent beam. Before failure, however, the concrete developed many cracks and flexural resistance was confined almost entirely to the timber. The diagram for Beam 5-B is fairly typical for Beams 2, 3, 5, 12, 13, and 14, while the one for Beam 6-B is typical for Beams 1, 4 and 6 to 11.

12. EFFECTIVENESS OF VARIOUS SHEAR CONNECTIONS

Table 4 shows the average shearing resistance developed by an individual shear connection of each type, based on the calculated maximum shearing stress on the horizontal area covered by one unit, and considering the bond between the concrete and creosoted timber to be negligible.

It is obvious that if the junction of timber and concrete fell on a single horizontal plane, coinciding with the neutral axis of the cross-section, the maximum shearing stress would be 1½ times the average shearing stress as in a homogeneous rectangular beam. Calculations indicate that the centroid of the composite section is 4.85 in. below the top of the beam, and the centroid of the bonding timber-concrete surfaces is only 0.45 in. lower. Hence the maximum shearing stress at the neutral axis is, with negligible error, 1½ times the average shearing stress, and the shearing stress at the tops of the 2 by 6 and 2 by 8 timbers is only about 1 per cent less than the maximum value. Hence the maximum value of 1.5 V/A is used in all computations of shears carried by shear connectors, where V is the external shear and A the area of actual cross-section.

In general, this value of the shearing stress should not be applied to all possible composite sections, such as T-beams or members in which

TABLE 4—SHEARING RESISTANCE DEVELOPED IN COMPOSITE BEAM TESTS

Beam No.	Kind of Shear Connection	Shear Unit	Max. Beam Load, P, Lb.	Max. Horizontal Shear*			Weight of Steel Conn. Unit, Lb.
				Stress Lb. per Sq. In.	Load per Shear Conn., Lb.	Load per Lb. of Steel Conn., Lb.	
0	None	....	50,025	...	....	....	....
1	Tri. pl., 12 ga., vert.	1 pl.	69,050	376	5750	28,750	.200
2	Tri. pl., ¼-in., vert.	1 pl.	62,450	339	5200	10,950	.475
3	Tri. pl., ⅜-in., vert.	1 pl.	64,475	366	5600	7,850	.712
4	Tri. pl., ¼-in., incl. 15 deg.	1 pl.	50,300	273	4180	8,800	.475
5	Tri. pl., ¼-in., vert., 60-d. spikes	1 pl. 1⅓ spk.	75,110	408	6250	10,420	.600
6	R.R. cut spikes, vert.	1 spk.	76,875	417	4800	5,910	.818
7	R.R. cut spikes, incl. 45 deg.	1 spk.	62,375	339	7020	8,590	.818
8	Sloped daps, 1 x 6-in.	1 dap†	51,960	282	6500	.....	....
9	Sloped daps, 1 x 6-in., 60-d. spikes	1 dap, 2 spk.	62,975	342	7850	7,200**	.188
10	Lag screws, 6 x ⅝-in., vert.	1 lag scr.	62,125	337	4650	9,080	.512
11	Lag screws, 8 x ⅝-in., incl. 45 deg.	1 lag scr.	71,815	390	8070	12,300	.655
12	Tri. pl., 12 ga., vert., 60-d. spikes	1 pl., 1⅓ spk.	68,160	370	5670	17,450	.325
13	Tri. pl., ¼-in., vert., 60-d. spikes	1 pl., 1⅓ spk.	69,100	375	5750	9,580	.600
14	Tri. pl., ⅜-in., vert., 60-d. spikes	1 pl., 1⅓ spk.	74,000	402	6150	7,350	.837
15	Tri. pl., 12 ga., vert., 60-d. spikes	1 pl., 1⅓ spk.	68,300	371	5680	17,480	.325

*Maximum horizontal shearing stress computed as $\frac{3V}{2\,bd} = \frac{P}{184}$, psi. Horizontal shear load per shear connection is product of shearing stress and area covered by one connection unit.
†Dap furnishes vertical bearing face, 1 in. high, 1⅝ in. wide.
**Increase over load carried on Beam 8.

the neutral axis and the junction plane are far apart. For such cases, the section may be reduced to an equivalent section of homogeneous material and the usual analysis of horizontal shearing stress made to determine the intensity of shear existing at the junction of the two materials.

The horizontal shear carried by the various connections at the maximum load does not necessarily indicate their full capacity, since most of the beams failed by tension in the timber, compression in the concrete, or in elements other than the shear units. The values listed for the shear units are therefore probably conservative in most cases.

Considering the triangular plate units, in Beams 1, 2 and 3, there is no consistent difference due to the plate thickness. In the corresponding Beams 12, 13 and 14, with 60-d. spikes added, there was a slight increase in shears carried with increase in plate thickness. However, in Beam 5, with ¼-in. plates and spikes, a still higher shearing resistance was obtained. Hence, the effect of plate thickness appears uncertain and probably negligible. The vertical position of the plates does seem important, however, since the results from Beam 4, with inclined plates, are unusually low.

The results from Beams 6 and 7, with railroad spikes and Beams 10 and 11, with lag screws, confirm the preliminary tests which showed roughly a 60 per cent advantage for the unit inclined at 45 degrees over the one placed vertically. The location of some of these steel units in the grooves of the laminated timber base was probably not too good, but the units as a group gave fairly satisfactory service.

Beam 8, employing sloping daps, and Beam 9, with daps and spikes, gave fair results. Failure occurred in the daps by shear of both timber and concrete, combined with tension in the timber. In Beam 8, a bearing pressure of 4000 psi was evidently developed on the vertical faces of daps. In Beam 9, with two spikes to each dap, an increase in shear over that for Beam 8, of 1350 lb. was developed.

The use of spikes was intended principally to prevent separation between concrete and timber layers. The amount they contributed to shearing resistance can only be determined indirectly from a comparison of the results of Beams 8 and 9, 1 and 12, 2 and 13, and 3 and 14, which also involve other variations. However, the average resistance of a 60-d. spike, as found from such comparisons is 440 lb. and this may be considered a rough approximation.

A comparison that takes on significance from the present shortage of steel may be made on the basis of shear developed per pound of steel per shear connection. This comparison has obvious weaknesses; it does not permit a comparison of timber daps with the other units, and it does not indicate, as a part of the cost of a unit, the cost of labor required to install it. However, Table 4 provides such a comparison. It is evident that the 12-gage triangular plates show much the highest values of shear per lb. of steel, followed by the 12-gage plates and spikes. The 8 by ⅝-in. lag screws are in third place and the railroad spikes, set vertically, are at the bottom of the list.

13. STRESSES AROUND SHEAR CONNECTIONS

A study has been made of the bearing and shearing stresses in timber and concrete in contact with the various steel shear units. Because of the irregular shapes involved, such calculations are admittedly far from precise.

For the triangular plates the areas in bearing on concrete and timber are known, but the intensity of bearing stresses is indeterminate. However, the *average* bearing stresses at failure are found to range from 900 to 1200 psi on the concrete and 3000 to 4200 on the timber. This includes the beams containing spikes, if allowance is made for the probable load carried by the spikes. These average stresses do not indicate any weakness due to bearing stresses produced by the units.

Lag screws, railroad and wire spikes are essentially dowels, but so short and irregular in shape that no accurate analysis is practicable. It is certain that high bearing stresses exist at the concrete and timber surfaces and are at least twice the average over the dowel length. Assuming a triangular bearing pressure diagram, maximum bearing stresses of 5500 psi in concrete and 3800 to 4200 psi in timber are indicated for the vertical railroad spikes and lag screws of Beams 6 and 10. For the same units inclined at 45 degrees the action is partly as a dowel and partly as a tension tie and the stress distribution is undoubtedly very complex.

Beams 8 and 9 contained "saw-tooth" daps with vertical bearing faces which produced bearing stresses in concrete and timber of 4000 psi or more; also horizontal shearing stresses at the base of the "saw-tooth" of 500 and 670 psi in timber and concrete, respectively. In several cases, failure due to such shearing stress was noted.

The bearing stresses mentioned were generally on areas well restrained by surrounding material, a condition in which bearing resistance well above the simple compressive strength of the material is to be expected. In general, the study does not indicate serious weakness of shear connections due to high bearing or shearing stresses in the surrounding material.

14. COMPARISON WITH RESULTS OF OTHER TESTS

A comparison of the test results with those mentioned at the beginning of this paper would seem useful. One series of tests was made at George Washington University and reported by J. F. Seiler; the other, made at Oregon State College, was reported by R. H. Baldock and C. B. McCullough.

The George Washington University tests were made on four composite beams and several "push-out" specimens. In the push-out specimens, made with laminated slabs of untreated yellow pine, those without shear connections developed bond values of 16 to 28 psi. Those with triangular plate shear developers, $\frac{3}{32}$-in. thick, developed gross shears per developer of 6880 lb. The beams, with a single load at the middle of a 12-ft. simple span, were similar to the beams of type 1; they had a width of 12.25 in., overall depth of 10.72 in. and were made with untreated yellow pine and concrete having a 7-day strength of 2950 psi.

The Oregon test beams were T-beams, with a concrete flange and solid timber stem, connected by various shear units. In Series 1, the beam

flanges were 15 in. wide and 6 in. deep; the stems 14 in. deep and 4 in. wide; in Series 2, the flanges were 24 in. wide and 6 in. deep; the stems 16 in. deep and 6 in. wide. All beams were tested on a 15-ft. span, with one-third point loading. The concrete had a compressive strength of 3630 psi, the timber a maximum fiber stress of 5810 psi.

The shear connectors included the following types: *Series 1*, A, ⅜ by 8-in. round spikes; B, rectangular daps in timber 1 in. deep, 5 in. long; C, combined daps and spikes; D, sections of 2½ in. steel pipe, 4½ in. long, set vertically into the timber 2½ in.; E, 4 by 4 by ¾-in. steel plates, driven into slots 1 in. deep in timber: *Series 2*, A, B, C, D and E, similar to Series 1, except that in D, 1½ in. pipe was used, and in E the plates were 6 in. wide.

The results from both groups of tests are given in Table 5 for comparison with the Illinois tests.

From the George Washington University tests, the average shear load per triangular plate was 7040 lb., neglecting bond. If a bond correction of even 16 psi were applied (since the timber was untreated) the shear load would show close agreement with the Illinois test results.

In Oregon tests, with pipe and plate dowels, the results were similar to those of the Illinois group with lag screws and railroad spikes, if the rough assumption is made that the maximum bearing stress is twice the average. A bearing stress of 4200 to 5000 psi in concrete and 3500 to 4000 psi in the timber is thus indicated for the pipe dowels, while for the plate dowels the corresponding stresses are 2600 psi in concrete and 7600 to 7800 in the timber. The latter value is rather high and it is of interest that the beams containing these plate dowels failed by horizontal shear in the timber at the base of the plates. Possibly there was a prying action of the plate dowels. The computed bearing stresses on the vertical faces of daps in the Oregon tests range from 3400 to 3700 psi, as compared with the value of 4000 psi for Beam 8 of the Illinois tests. The combination of daps and spikes was found effective in reinforcing the timber against horizontal shear at sections below the junction with the concrete, and this type of connection is stated to be both effective and economical in the Oregon report. The pipe dowels are also rated as very effective in minimizing slip and producing integral action of the beam, though rather expensive to install.

15. DESIGN CONSIDERATIONS

In applying the foregoing ultimate strengths of shear connections to design, consideration must be given to the way in which service loads are applied, to the conditions of exposure, estimated life of the structure and other factors. The factors of safety applied to the shear unit should be at least as great as for the timber and concrete. For use in floors of highway bridges under normal vehicular traffic and so detailed that the

TABLE 5—SUMMARY OF ULTIMATE RESISTANCE OF SHEAR DEVELOPERS

Tests at George Washington University, Oregon Highway Department, and University of Illinois.

Test	Beam No.	Type of Connection	Max. Shear Load on 1 Connection* Lb.	Remarks
G. W. U.	1, 2	Triangular plates	7,040	
	3, 4		7,040	
Oregon, Series 1 (Timber Stem, 4 x 14 in.)	1, 2	2½ in. pipes, 4½ in. long	14,610	2½ in. in timber, 2 in. in concrete
	3, 4	4 x 4 x ¾-in. plates	15,230	1 in. in timber, 3 in. in concrete
	5, 6	5 in. dap, 1 in. deep; 3 spikes, ⅜ x 8 in.	18,760 (on unit)	Daps across 4 in. width on beam
	13, 14	⅜ x 8 in. spikes	3,785	5 in. in timber, 3 in. in concrete
	15, 16	5 in. daps, 1 in. deep	14,720	On 10 in. length of beam
Series 2 (Timber Stem, 6 x 16 in.)	7, 8	1½ in. pipes, 4½ in. long	8,050	2½ in. in timber, 2 in. in concrete
	9, 10	⅜ x 8 in. spikes	3,880	5 in. in timber, 3 in. in concrete
	11, 12	5 in. daps, 1 in. deep	20,160	Dap across 6 in. width of timber
	17, 18	10 in. dap, 1 in. deep; 9 spikes, ⅜ x 8 in.	58,140 (on unit)	On 20 in. length of beam
	19, 20	6 x 4 x ¾ in. plates	23,410	1 in. in timber, 3 in. in concrete
Univ. of Ill.	1, 2, 3	Triangular plates	5,520	Av. of 3 thicknesses
	4	Tri. pl., at 15 deg.	4,180	
	5, 12, 13, 14, 15	Tri. pl., 60-d. spikes	5,900	On 1 plate and 1⅓ spikes
	6	R.R. spikes, vertical	4,800	4 in. in timber, 2½ in. in concrete
	7	R.R. spikes, at 45 deg.	7,020	3¾ in. in timber, 2¾ in. in concrete
	8	Daps, 1 in. deep, 6 in. long	6,500	Dap across 1⅝ in. width of timber
	9	Daps and 60-d. spikes	7,850	On 1 dap and 2 spikes
	10	Lag screws, vertical	4,650	3½ in. in timber, 2½ in. in concrete
	11	Lag screws, at 45 deg.	8,070	4½ in. in timber, 3½ in. in concrete

*Bond between concrete and timber neglected in computing these values.

timber portion of the floor is well protected from the weather, a factor of safety of 3.5 to 4 seems advisable. Such a factor, for the material used in these tests, should not produce undue bearing or shearing stresses.

Considering first the triangular plate unit, as most important in current construction practice, the average ultimate shear load for Beams 1, 2 and 3 is 5520 lb., that for Beams 5, 12, 13, 14 and 15, with spikes added, is 5900 lb. Applying a factor of safety of 3.75 gives design values of 1470

and 1575, for the two respective groups. While the spikes should always be used with the triangular plates to prevent uplift, they are generally used at much greater intervals than in these tests. Hence, it is recommended that, though the spikes should be used, they be disregarded as shear units and a value of 1500 lb. be used for the triangular plates, when materials of the quality tested are employed.

Application of a factor of safety of 3.75 to the various units tested thus provides the following values of design shear load per unit.

Triangular plates, 12 gage or thicker, with uplift spikes ..... 1500 lb.
Railroad spikes, 6½ by ⅝ in., vertical ..... 1280 lb.
Railroad spikes, 6½ by ⅝ in., inclined at 45 deg. ..... 1875 lb.
Lag screws, 6 by ⅝ in., vertical ..... 1240 lb.
Lag screws, 8 by ⅝-in., inclined at 45 deg. ..... 2150 lb.
Timber daps, 1070 psi. bearing stress on vertical surfaces
180 psi. shearing stress on horizontal surfaces of concrete
135 psi. shearing stress on horizontal surfaces of timber

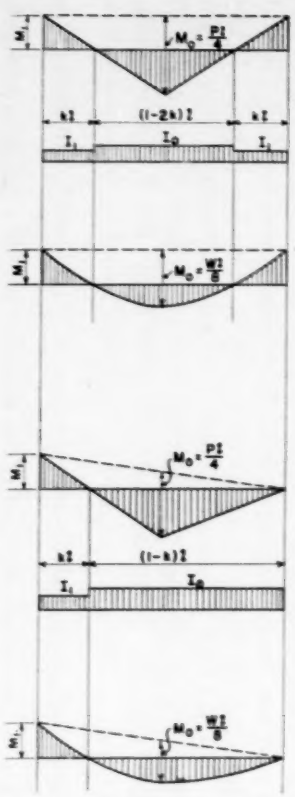
It is evident that the railroad spikes and lag screws serve the purpose of uplift spikes, but the spikes should be used in all cases with triangular plates or timber daps.

16. APPLICATIONS TO CONTINUOUS MEMBERS

The application of composite construction to continuous structures presents further problems. Obviously, in regions of negative moment, the concrete is placed in tension and the timber in compression. The logical procedure is to reinforce the concrete with steel, following usual reinforced concrete practice, and to stagger splices in the laminated timber, or to use other details to produce high effectiveness of timber section at points of high negative moment. In long structures, such as highway trestles, discontinuity at intervals should be introduced to provide the necessary expansion joints.

The determination of bending moments in continuous composite beams may be made by ordinary structural analysis, but allowance should be made for the difference in stiffness between regions of negative and positive moments. Differences of this sort may be represented by the value, I, of the transformed sections of the beam in the two regions.

As a starting point in the determination of moments, values of the critical bending moment in "fixed end" beams and in beams with one end fixed and one freely supported have been determined for uniform and concentrated loading, and are given in Fig. 8. For this purpose, it has been assumed that a constant moment of inertia I_1 exists in the region of negative moment, and that a different value I_o exists in the region of positive moment. In the solution for Cases II to IV, Fig. 8, the expression for the ratio, k, is complex, and a solution is found by inserting values of k and finding the corresponding values of $\frac{I_o}{I_1}$. Figure 9 contains curves which give the values of k and of the negative M_1 for a range of probable



I. Fixed Ends; Concentrated Load, P, at Midspan.

$$M_1 = \frac{M_o}{1+\sqrt{\frac{I_o}{I_1}}} \qquad (1)$$

$$M_1 = 2k M_o \qquad (2)$$

II. Fixed Ends; Uniform Load, W.

$$M_1 = 4k(1-k)M_o \qquad (3)$$

$$M_1 = \frac{M_o}{0.92 + 0.58\sqrt{\frac{I_o}{I_1}}} \text{ (approx.)} \qquad (4)$$

$$\frac{I_1}{I_o} = \frac{2k^2(3-4k)}{(1-2k)^3} \qquad (5)$$

III. One End Fixed, One Free; Concentrated Load, P, at Midspan.

$$M_1 = M_o\left(\frac{2k}{1-k}\right) \qquad (6)$$

$$\frac{I_1}{I_o} = \frac{2k}{1-2k}\left(\frac{6k-2k^2}{3-5k+2k^2}\right) \qquad (7)$$

IV. One End Fixed, One Free; Uniform Load, W.

$$M_1 = 4k M_o \qquad (8)$$

$$\frac{I_1}{I_o} = \frac{k^2(k^2-4k+6)}{(1-k)^4} \qquad (9)$$

Fig. 8—Moments in continuous beams with variable moments of inertia

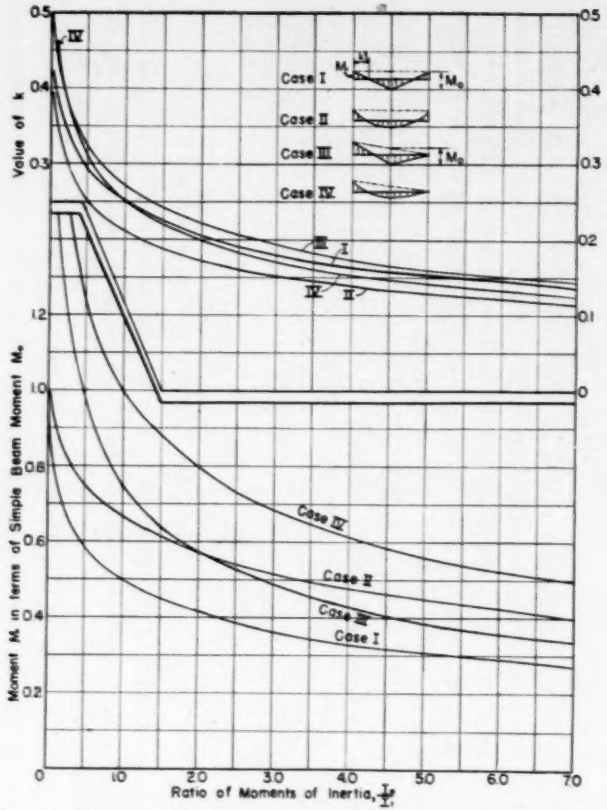


Fig. 9—Diagram of moments and distances to points of inflection in continuous beams

values of $\frac{I_o}{I_1}$. From Fig. 9, values of the bending moments and the location of the point of inflection may be found for the values of $\frac{I_o}{I_1}$ applying to the section assumed for a given beam design. The section may then be investigated to see if it satisfies the bending moments found.

The solution of a multiple span continuous beam with varying moments of inertia is a rather difficult problem. However, the curves of Fig. 9 should indicate to the designer how much departure may be expected from the usual case of a beam of uniform section. If the moment distribution method of analysis were applied, exact carry-over and distribution factors would be hard to determine, but if the ratio $\frac{I_o}{I_1}$ does not depart too far from unity, it may be sufficiently accurate to use the fixed end moment values from Fig. 9, and to distribute unbalanced moments by the conventional carry-over and distribution factors.

17. CONCLUSIONS

The principal conclusions summarized from this study are:

1. The composite timber-concrete beams included in these tests exhibited, in most cases, fairly satisfactory structural action. The degree of integral action secured with various types of shear connections, while not perfect, was sufficiently good to encourage the use of this kind of member. In view of the small amount of steel required, the possibility of using small dimensioned timber and the advantages of a top surface of concrete to resist wear and to give substantial weather and fire protection to the timber, the use of composite construction for heavy duty floors for bridges, piers and similar structures seems especially logical at this time.

2. Of the shear connection studied, the triangular plate units, combined with wire spikes, appeared to be the most satisfactory in producing integral beam action, small slips and high strength and stiffness. The 12-gage plates were by far the most economical of the plates tested and were evidently thick enough to give the necessary structural strength. This type of unit is not difficult to install in practical construction; however, it should always be used in combination with "uplift" spikes, or their equivalent, because of the inability of the plates alone to tie the two elements of the composite member together.

3. Of the other shear units used the railroad spikes, lag screws, and timber daps with spikes produced fairly good results, though slip between timber and concrete was appreciable and there was considerable departure from integral action of the beam. Timber daps alone gave rather poor results, and failure occurred in some cases in the shear connection.

4. The stiffness of the better types of composite beams tested was slightly less than that computed for a solid timber beam of the same

dimensions and quality of material; the strength was considerably less, but this feature is probably offset by some of the advantages of the composite structure.

5. The variation in volume change (shrinkage or expansion) of the two materials of the composite beam, measured in these tests over a 2½ year period, apparently had little or no effect upon the strength of the beams, as compared to beams tested 28 days after fabrication. There was no indication of separation of the two elements of the beams and the stiffness and integral action were, if anything, slightly superior for the beams 2½ years old.

6. The repeated load tests on the beams of type 16 furnish a limited amount of data indicating that under a considerable number of repetitions of a design or working load no appreciable permanent deformation will occur to interfere with the normal structural behavior of the members. With repeated application of a much higher load (roughly two and one-half times the working load) definite permanent sets were developed, but even these did not produce any serious weakening of the member, nor did they interfere with the satisfactory structural behavior of the beam when rapid loading to failure was resumed.

7. From these tests, allowable values for various types of shear connections have been determined, and are believed to be suitable for safe, conservative design of composite beams and slabs. Especially with treated timber, it seems necessary to neglect any bond or adhesive resistance between the timber and concrete, but to design the shear connection to carry the full maximum horizontal shear.

8. The test beams apparently were well proportioned to utilize the potential strength of both the concrete and timber used, since in many beams it seemed to be a matter of chance whether initial failure occurred by compression in the concrete or tension in the timber.

9. While the test beams were tested as simple spans, the type of construction seems easily applicable to continuous spans. Methods of calculation of moments in continuous members are included herein to facilitate design procedure.

18. ACKNOWLEDGMENT

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Balanced Design for Reinforced Concrete*

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SYNOPSIS

The question of factor of safety on reinforced concrete design is now foremost in engineers' minds. This has been brought about by shortage of reinforcing steel and the issuance of directives increasing the permissible steel stresses. In this paper the authors question the practice of a uniform factor of safety on dead and live load and suggest a method of design which they believe would permit a better distribution of reinforcing steel than could be obtained by a flat increase in working stresses, and at the same time reduce the required amount of reinforcing steel without a reduction in the factor of safety.

Reinforced concrete is a relatively new material, and while progress has been startling, thinking has been hampered by a natural tendency to seek precedent from experiences with older materials whose tensile and compressive strengths are more nearly equal. Long before reinforced concrete had gained its present position as a structural material, structures were designed on an ultimate strength basis and a "factor of safety." Prof. Clyde T. Morris has this to say on the subject**:

Forty years ago steel structures were designed with a, so called, "factor of safety." This was usually based on the ultimate tensile strength of the material and had little real significance, as the ultimate capacity of a structure designed with a nominal factor of safety of 4 was seldom more than twice the design load. In those days the factor of safety was in reality a factor of ignorance and was used largely by salesmen in dealing with big officials who had little or no engineering knowledge. Due to this fact the term fell into disrepute and allowable working stresses for design were substituted. Now, the term factor of safety is seldom used. In dealing with a material such as steel which has a straight line relationship between stress and strain and for which the properties do not change with time and conditions of exposure, the permissible unit stress method of design is probably preferable to the use of a factor of safety on the ultimate strength. *But for concrete structures there seems to be considerable merit in the suggestion that designs be made for ultimate loads with suitable factor of safety.*

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**In a discussion, p. 498 - 1, ACI Jl. Supplement, Sept.-Oct. 1937; *Proceedings* V. 33, of C. S. Whitney's paper, "Design of Reinforced Concrete Members under Flexure or Combined Flexure."

The practice of specified unit stresses for $DL + 1LL$* with some materials is easily understood. The dead to live load ratio is a relatively small number and varies over a narrow range, say, from 0.2 to 0.5. Because of the weight of concrete construction the ratio may be from about 0.4 to 2 or 3. What is more important, this ratio may vary over this wide range with different members of the same structure—even at different points in the same member. If the ratio of $\frac{DL}{LL}$ were positive and constant for all members or all points in a member in a given structure, design based on permissible stresses would be satisfactory for all materials, but for a material such as concrete where this ratio varies over such a wide range it definitely puts a question on our present practice.

Anyone investigating an old structure for load rating purposes must be greatly impressed by the great difference in the live load carrying capacities of the different members in a structure. Engineers have long since recognized the inadequacy of our present design methods and for years have been devising ways and means to overcome this deficiency. This is illustrated by the present practice of some highway bridge departments that are reported to be now designing their floor systems for weight of truck of, say, H20 and the main girders for a different loading, H15.

Evidence of a growing dissatisfaction with our present practice can be seen in the "Standard Specifications for Highway Bridges and Incidental Structures," adopted by the American Association of State Highway Officials in 1931. On page 185 of this specification is given, under the heading Reinforcement,

steel reinforcement
tension
for live load and lateral forces — 16000 psi.
for dead load, temperature and shrinkage — 24000 psi.

In this same specification a limitation is placed on the permissible stress for dead load plus 2 live loads. This committee did two things of note: (1) It specified a different stress for dead load and live load; this was a definite attempt to balance the design. (2) It established the stress produced by dead load and two live loads as the limiting stress. The above specification was a tacit admission of the inadequacy of present methods of design.

Further evidence of the tendency to apply to reinforced concrete methods applicable to other and more homogeneous materials can be seen in our present practice of assuming a straight line variation of

*DL and LL are used to denote stress contributed by dead load and live load plus impact. In general, DL and LL will be used to represent the stress these loadings produce, be it moment, shear, or bond.

stress within the section. This was pointed out by L. J. Mensch in the JOURNAL, American Concrete Institute, Dec., 1914, p. 28.

OVERLOAD FACTORS

The present scheme of design in a sense assumes the same factor of safety, or what might better be termed an overload factor, for dead and live load. In Professor Morris' discussion previously quoted, he suggests the value of 3 applicable both to dead and live load. If we carry Professor Morris' thought further we might write $3DL + 3LL = 40000$, and we arrive at a working stress of 13300. Certainly, after years of experience with lesser factors and in view of the constant improvement in materials technology the profession would hesitate to adopt these values.

Is it logical to require the same overload factor for dead as for live load? Dead load is a quantity which can be determined with a high degree of accuracy. It is always present and is a static thing. Live load as used in design, at best, is an estimate and more often a general specification requirement which may or may not be adequate even at the time of design. Certainly it can vary from time to time as occupancy changes during the life of the structure. If a bridge, the trucks may become heavier with time. Unexpected emergencies may arise which subject the structure to overload conditions. For these reasons it would seem desirable to have different overload factors on the different types of loading.

The purpose of any structure is to carry economically and safely any anticipated loads to which that structure may be subjected during a reasonable life expectancy. To do this each material used in fabrication of the structure, whether it be wood, steel, or reinforced concrete, should be treated individually. If any two or more materials have properties which make it proper to design to advantage by one procedure, well and good. If, however, any material has distinctive or intrinsic properties which require special or individual treatment, a code could and should be written to take full advantage of those characteristics.

SAFETY OF STRUCTURES

Whenever engineers discuss the subject of the desire for economy of structures, the first inclination is to increase working stresses irrespective of the type of structure, weight of structure, character of live load to be carried, span lengths, materials involved, or the conditions to be met during or after construction. It does not follow that increasing the working stresses necessarily makes for economy in reinforced concrete structures, nor does it follow that the greatest saving of materials may thus be made. It will be shown later that generally no appreciable saving can

be obtained by merely increasing the allowable stresses without reducing the factor of safety correspondingly. Why reduce the factor of safety to secure economy if the same end can be accomplished by eliminating the unusable excess strength obtained by conventional design?

If we have a relatively light structure and a heavy live load, should present working stresses be greatly increased? Under these conditions most of the strength of the member is used in carrying the live load and any error in estimating the live load is reflected almost directly in increased stresses. In this case the important thing is live load. If, on the other hand, we have a heavy structure carrying a comparatively light live load, may we not be justified in permitting higher stresses? In this case the important factor is dead load. This we can estimate with a high degree of accuracy. It is a fixed and very real thing. Any error we make in our estimate of live load is not proportionately reflected in the resulting stresses.

As designs are made by conventional methods we say

$DL + LL = f_s$, where f_s is the steel stress used in conventional design. How many live loads, F_L, can be placed on the structure before the yield point stress of the steel, f_{yp}, is reached? Or

$DL + F_L LL = f_{yp}$, where f_{yp} is the yield point stress of reinforcement. From these two equations

$$F_L = \frac{f_{yp}}{f_s}\left(\frac{DL}{LL} + 1\right) - \frac{DL}{LL}$$

assuming $f_{yp} = 40000$

$f_s = 20000$

then $$F_L = 2 + \frac{DL}{LL} \quad \ldots\ldots\ldots\

PROPOSED METHOD OF DESIGN

A design can be made on an ultimate strength basis as follows:

$$F_D DL + F_L LL = f_{yp} \quad \ldots\ldots (2)^*$$

where F_D and F_L are factors of safety on dead and live load stress respectively.

The assigning of values to F_D and F_L will require much study and sound judgment by responsible and experienced designers. Certainly F_D does not need to be as great as F_L and it would seem that a value of unity might be assigned to F_D and an F_L chosen large enough amply to safeguard the structure provided that further consideration be given to the condition where only dead load is being carried or where dead load alone might cause excessive stresses. This situation will be discussed later.

The only value of F_D which will give a constant factor of safety for the live load in all portions of a structure is unity. Assume this value to illustrate the method and to indicate the effects of its application, we may write

$$DL + F_L LL = f_{yp} \quad \ldots\ldots (2a)$$

or

$$\frac{DL}{F_L} + LL = \frac{f_{yp}}{F_L} = f_s \quad \ldots\ldots (2b)$$

Then dividing Equation (2a) by (2b) we have

$$\frac{DL + F_L LL}{\frac{DL}{F_L} + LL} = F_L$$

assume $f_{yp} = 40000$

$F_L = 2$

then $f_s = 20000$

As can be seen from Equation (2b), ultimate design can be accomplished in this case by designing for

$0.5DL + LL$

using f_s at 20000 psi.

Had we assumed $F_L = 2.5$ rather than 2.0, from Equation (2b) we would obtain ultimate design by using

$$0.4\ DL + LL, \text{ using as } f_s \frac{40000}{2.5} = 16000.$$

It should be noted here that the actual stress f'_s at any point depends on the ratio of $\frac{DL,}{LL}$ and if it is desired to know the actual stress for dead

*This equation and those that follow are based on the assumption that the useful strength of reinforced concrete is exhausted when $f_s = f_{yp}$.

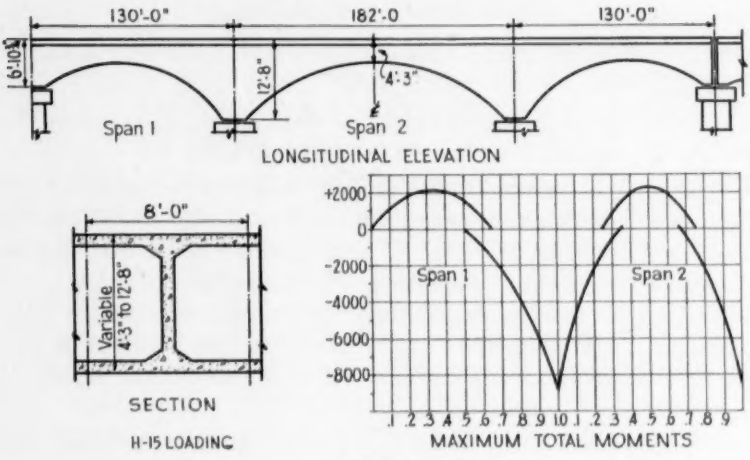

Fig. 1

load plus live load it may be computed from the equation

$$f'_s = f_s \frac{DL + LL}{kDL + LL}$$

where $$k = \frac{1}{F_L}$$

for n live loads

$$f'_s = f_s \frac{DL + nLL}{kDL + LL} \qquad (3)$$

The procedure here outlined has been applied to the analysis of the decks of two structures of rather extreme loading conditions. Fig. 1 shows the profile, cross section and general dimensions of a hollow box girder symmetrical about the center line carrying the current A.A.S.H.O. loading for an H15 truck train with specified impact (a rather light live load as compared to dead load). The structure was designed as a three-span continuous bridge acting on roller bearings as supports. The stresses used in the conventional design were $f_c = 1200$ psi, $f_s = 20000$ psi.

Table 1 shows the variation in designs obtained by holding f_c and f_s constant and assuming $k = 1.0$ (conventional design); 0.7; 0.6; and 0.5.

The column marked $\frac{DLM}{LLM}$ shows the great variation in this ratio from point to point.

It will be especially noted that as k is reduced the volume of both concrete and steel is reduced and stresses for dead load only slowly increase.

TABLE 1—STRESSES IN HEAVY STRUCTURE

$kDL + LL$

$f_c = 1200$ $f_s = 20000$

Point	$k = 1.0$					$k = 0.7$				$k = 0.6$				$k = 0.5$			
	DLM	LLM*	$\frac{DLM}{LLM}$	$DL+2LL=f_s$	$DL=f_s$	DLM	$\frac{DLM}{LLM}$	$DL+2LL=f_s$	$DL=f_s$	DLM	$\frac{DLM}{LLM}$	$DL+2LL=f_s$	$DL=f_s$	DLM	$\frac{DLM}{LLM}$	$DL+2LL=f_s$	$DL=f_s$
$.20L_1$	1092.5	671	1.63	27600	12400	890	1.33	34500	13800	841	1.25	37100	14300	834	1.24	40000	15300
$.33L_1$	1154.1	911	1.27	28800	11800	981	1.08	35200	12300	927	1.02	37400	12700	881	.97	40000	13000
$.50L_1$	523.5	992	.53	33100	6900	445	.45	37200	6800	421	.43	38600	6700	400	.40	40000	6700
$.60L_1$	−214.9	844	−.26	46800	.....	−183	−.22	42000	.....	−73	−.21	40900	.....	−164	−.19	40000	.....
$.60L_1$	−214.9	−814	.26	35800	4200	−183	.22	38400	3900	−73	.22	39000	3800	−164	.20	40000	3700
$.80L_1$	−2565.6	−1210	2.12	26400	13600	−2181	1.80	33600	15900	−2061	1.70	36600	16800	−1958	1.62	40000	18000
$.90L_1$	−4257.4	−1558	2.73	25400	14500	−3618	2.32	32900	17700	−3420	2.20	36200	18900	−3249	2.09	40000	20400
$1.00L_1$	−6377.2	−2111	3.02	25000	15000	−5420	2.57	32600	18300	−5122	2.42	36000	19800	−4867	2.31	40000	21400
$.20L_2$	−1266.7	−862	1.47	28100	12000	−1077	1.25	34700	13300	−1017	1.18	37200	13800	−967	1.12	40000	14400
$.30L_2$	160.9	−582	−.24	47600	.....	137	−.24	42200	.....	129	−.22	41000	.....	123	−.21	40000	.....
$.30L_2$	160.9	662	.24	36100	3900	137	.21	38500	3600	129	.20	39300	3500	123	.19	40000	3400
$.50L_2$	1245.9	984	1.27	28800	11200	1059	1.08	35200	12300	1001	1.02	37500	12600	951	.97	40000	13000
Relative Quantities Both Concrete and Steel					100%				85%				80.3%				76.3%

*LLMs same for all designs.

Further, it can be seen that the stresses in columns marked $DL + 2LL = f_s$ show less variation from point to point in the structure and, except for the points $0.6L_1$ and $0.3L_2$, gradually increase until they all reach 40000 psi when $k = 0.5$. The stresses at the $0.6L_1$ and $0.3L_2$ points decrease and reach 40000 psi when $k = 0.5$.

Tables No. 2, 3 and 4 show the results of designing the same structure by conventional methods for unit stresses of $f_c = 1200$, $f_s = 22000$ and 24000; $f_c = 1050$, $f_s = 22000$ and 24000; and $f_c = 975$, $f_s = 22000$ and 24000, as compared with a conventional design of $f_c = 1200$ and $f_s = 20000$ psi.

Two points brought out by these designs are of particular interest: (1) The savings in steel by lowering the concrete stress and increasing the steel stress are not large in the case of a heavy structure with a comparatively light live load. (2) In the zone between points "A" and "B", Fig. 2, any overloading causes very high stresses with the conventional type of design.

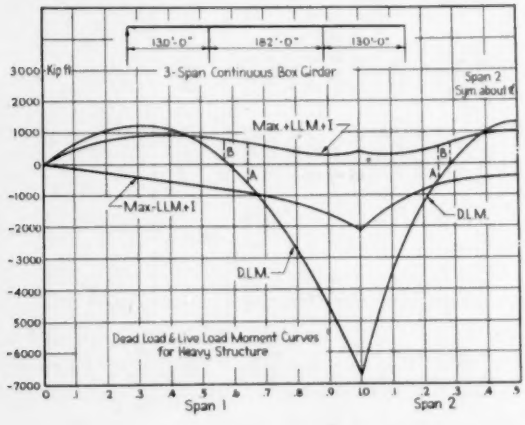

Fig. 2

The type of structure illustrated in these tables (heavy dead load and light live load) is greatly influenced by the dead load and an increase in working stresses would not ordinarily be objectionable to most designers. Nevertheless, in the zone "A"-"B", Fig. 2, special investigations should be made when usual design procedures are followed or when any value of k greater than 0.5 is used. Merely designing for a combined dead and live load moment curve may not be enough.

Table 5 gives a recapitulation of quantities for various combinations of stresses and methods discussed above.

Fig. 3 shows the profile, cross section and general dimensions of a crane girder, symmetrical about the center line, carrying two 150-ton cranes as

TABLE 2—HEAVY STRUCTURE, LIGHT LIVE LOAD—COMPARISON OF STRESSES AND MATERIAL QUANTITIES

Designed for *DL & LL*

Point	Design basis $f_c = 1200$, $f_s = 20000$				Design basis $f_c = 1200$, $f_s = 22000$			Design basis $f_c = 1200$, $f_s = 24000$		
	DLM	*LLM**	$\frac{DLM}{LLM}$	$DL+2LL=f_s$	*DLM*	$\frac{DLM}{LLM}$	$DL+2LL=f_s$	*DLM*	$\frac{DLM}{LLM}$	$DL+2LL=f_s$
$.20L_1$	1092.5	671	1.63	27600	1108.0	1.65	30300	1131.0	1.69	32900
$.33L_1$	1154.1	911	1.27	28800	1170.3	1.29	31600	1194.6	1.31	34400
$.50L_1$	523.5	992	.53	33100	530.9	.54	36300	541.9	.55	39500
$.60L_1$	−214.9	844	−.26	46800	−218.0	−.26	51700	−222.5	−.26	56500
$.60L_1$	−214.9	−814	.26	35800	−218.0	.27	39300	−222.5	.27	42800
$.80L_1$	−2565.6	−1210	2.12	26400	−2601.7	2.15	29000	−2655.8	2.19	31500
$.90L$	−4257.4	−1558	2.73	25400	−4317.2	2.77	27800	−4407.1	2.83	30300
$1.00L_1$	−6377.2	−2111	3.02	25000	−6466.6	3.06	27400	−6601.3	3.13	29800
$.20L_2$	−1266.7	−862	1.47	28100	−1289.5	1.49	30800	−1311.2	1.52	33500
$.30L_2$	160.9	−582	−.24	47600	163.2	−.28	52600	166.6	−.29	57700
$.30L_2$	160.9	662	.24	36100	163.2	.25	39700	166.6	.25	42900
$.50L_2$	1245.9	984	1.27	28800	1263.4	1.29	31700	1289.7	1.31	33800
Relative Quantities	Concrete 100% Steel 100%				Concrete 101.5% Steel 90.6%			Concrete 103.5% Steel 83.6%		

*Live load same for all designs.
Steel ratios are based on main steel required as T-beams.

TABLE 3—HEAVY STRUCTURE, LIGHT LIVE LOAD—COMPARISON OF STRESSES AND MATERIAL QUANTITIES

Designed for *DL* & *LL*

Point	Design basis $f_c = 1200$, $f_s = 20000$				Design basis $f_c = 1050$, $f_s = 22000$			Design basis $f_c = 1050$, $f_s = 24000$		
	DLM	*LLM**	$\frac{DLM}{LLM}$	$DL+2LL=f_s$	*DLM*	$\frac{DLM}{LLM}$	$DL+2LL=f_s$	*DLM*	$\frac{DLM}{LLM}$	$DL+2LL=f_s$
$.20L_1$	1092.5	671	1.63	27600	1298.2	1.93	29500	1329.4	1.98	32100
$.33L_1$	1154.1	911	1.27	28800	1371.4	1.50	30800	1404.3	1.54	33400
$.50L_1$	523.5	992	.53	33100	622.2	.62	35500	637.0	.64	38600
$.60L_1$	−214.9	844	−.26	46800	−255.4	−.30	53500	−261.5	−.31	59700
$.60L_1$	−214.9	−814	.26	35800	−255.4	.31	38700	−261.5	.32	42200
$.80L_1$	−2565.6	−1210	2.12	26400	−3048.8	2.52	28200	−3121.9	2.58	30700
$.90L_1$	−4257.4	−1558	2.73	25400	−5059.0	3.24	27200	−5180.5	3.32	29500
$1.00L_1$	−6377.2	−2111	3.02	25000	−7578.0	3.59	26800	−7759.9	3.67	29100
$.20L_2$	−1266.7	−862	1.47	28100	−1505.2	1.75	30000	−1541.4	1.79	32600
$.30L_2$	160.9	−582	−.24	47600	191.2	−.33	54500	195.8	−.34	60100
$.30L_2$	160.9	662	.24	36100	191.2	.29	39100	195.8	.30	42500
$.50L_2$	1245.9	984	1.27	28800	1480.5	1.50	30800	1516.0	1.54	34800
Relative Quantities	Concrete 100% Steel 100%				Concrete 118.8% Steel 91.2%			Concrete 121.7% Steel 84.0%		

*Live load same for all designs.
Reinforcement % computed on basis of main steel.

TABLE 4—HEAVY STRUCTURE, LIGHT LIVE LOAD—COMPARISON OF STRESSES AND MATERIAL QUANTITIES

Designed for *DL* & *LL*

Point	Design basis $f_c = 1200$, $f_s = 20000$				Design basis $f_c = 975$, $f_s = 22000$			Design basis $f_c = 975$, $f_s = 24000$		
	DLM	*LLM**	$\frac{DLM}{LLM}$	$DL+2LL=f_s$	*DLM*	$\frac{DLM}{LLM}$	$DL+2LL=f_s$	*DLM*	$\frac{DLM}{LLM}$	$DL+2LL=f_s$
$.20L_1$	1092.5	671	1.63	27600	1434.0	2.14	29000	1523.0	2.18	31300
$.33L_1$	1154.1	911	1.27	28800	1515.0	1.66	30300	1609.0	1.69	32700
$.50L_1$	523.5	992	.53	33100	687.5	.69	35000	730.0	.71	37800
$.60L_1$	−214.9	844	−.26	46800	−282.0	−.33	55000	−300.0	.34	61300
$.60L_1$	−214.9	−814	.26	35800	−282.0	.35	38300	−300.0	.35	41500
$.80L_1$	−2565.6	−1210	2.12	26400	−3368.0	2.78	27800	−3577.0	2.83	30000
$.90L_1$	−4257.4	−1558	2.73	25400	−5589.0	3.58	26800	−5936.0	3.65	29000
$1.00L_1$	−6377.2	−2111	3.02	25000	−8373.0	3.96	26400	−8892.0	4.04	28600
$.20L_2$	−1266.7	−862	1.47	28100	−1663.0	1.93	29500	−1776.0	1.96	31800
$.30L_2$	160.9	−582	−.24	47600	211.3	−.36	56600	224.0	−.37	63000
$.30L_2$	160.9	662	.24	36100	211.3	.32	38600	224.0	.33	42000
$.50L_2$	1245.9	984	1.27	28800	1635.0	1.66	30200	1737.0	1.69	32700
Relative Quantities	Concrete 100% Steel 100%				Concrete 131.3% Steel 92.3%			Concrete 139.4% Steel 85.7%		

*Live load same for all designs. Reinforcement % computed on basis of main steel.

TABLE 5—HEAVY STRUCTURE LIGHT LIVE LOAD

Comparison of Concrete and Steel Quantities

	Conventional Design						Balanced Design			
Designed for $f_c = 1{,}200$, $f_s = 20{,}000$	$f_c = 1{,}200$, $f_s = 22{,}000$	$f_c = 1{,}200$, $f_s = 24{,}000$	$f_c = 1{,}050$, $f_s = 22{,}000$	$f_c = 1{,}050$, $f_s = 24{,}000$	$f_c = 975$, $f_s = 22{,}000$	$f_c = 975$, $f_s = 24{,}000$	$k = 0.75$	$k = 0.7$	$k = 0.6$	$k = 0.5$
Concrete = 100%	101.5	103.5	118.8	121.7	131.3	139.4	88.2	85	80.3	76.3
Steel = 100%	90.6	83.6	91.2	84.0	92.3	85.7				

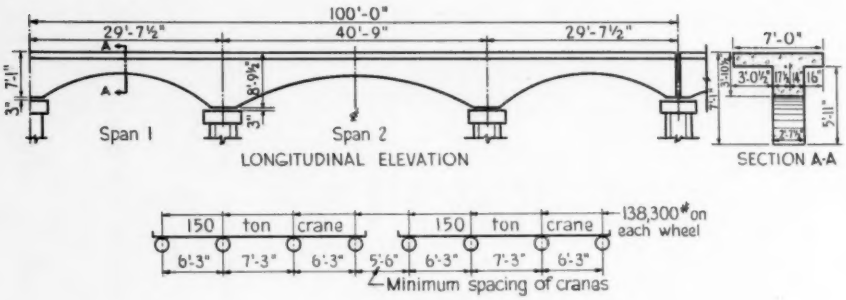

Fig. 3

shown in the loading diagram. This structure was designed as a three-span continuous structure acting on knife edge supports. The stresses used in the conventional design were $f_c = 1200$ psi, $f_s = 20000$ psi.

Table 6 shows the variation in the designs obtained using $f_c = 1200$, $f_s = 20000$ and assuming $k = 1.0$ (conventional design) and using $f_c = 1050$, $f_s = 20000$ and assuming $k = 0.5$ (balanced design.) It will be noted from Table No. 6 that the column maked $DL + LL = f_s$ shows very nearly constant values which are only slightly greater than the design stress used in the conventional method. A similar column was not computed for the heavy structure, but had this been done values of over 30000 psi would have been found. This indicates the small influence of dead load in the light structure.

Tables 7, 8 and 9 are comparable to Tables 2, 3, 4 previously discussed for the heavy structure.

Fig. 4 shows the moment curves for the light structure. It will be noted that there are no points on these curves which compare with point "A" in Fig. 2 and hence there is no place in this structure where the theoretical factor of safety reduces to unity. In the zone between "B" and toward the intermediate support this factor is, however, something less than 2.

As designs are generally made, the dead and live load moment curves are combined and steel placed accordingly. The designer generally assumes the moments near the point of counter-flexure are small and are taken care of by running a nominal amount of steel through to the support. Both the moment curves and tables disprove this and indicate clearly the need for special consideration in this zone.

Table 10 shows a tabulation of materials for all of the design conditions shown in the tables above referred to for the light structure and heavy load.

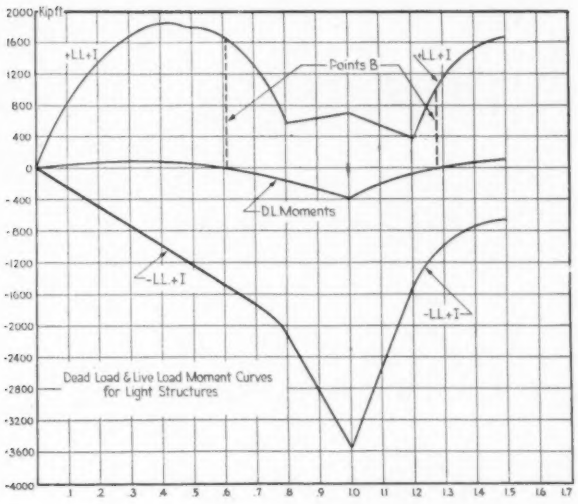

Fig. 4

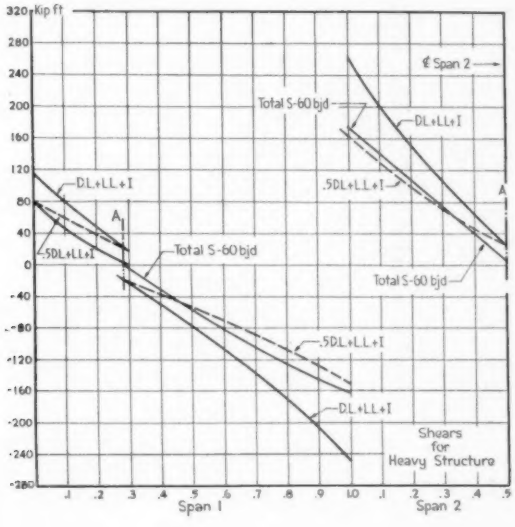

Fig. 5

SHEAR AND BOND

In Fig. 5 are shown the shear diagrams for the structure of Fig. 1. Two values have been plotted, one for full dead load plus live load and impact according to the conventional design, i.e., $k = 1.0$; one for a balanced design using a k value of 0.5.

It should be recalled that balanced design is actually ultimate design, and in this case for one dead load and two live loads with impact. Numerous tests have shown that shear reinforcement does not participate fully until the concrete has failed by diagonal tension when, theoretically,

TABLE 6—STRESSES IN LIGHT STRUCTURE

$kDL + LL$

Point	$f_c = 1200$, $f_s = 20000$, $k = 1.0$				$f_c = 1050$, $f_s = 20000$, $k = 0.5$			
	DLM	LLM*	$\frac{DLM}{LLM}$	$DL+2LL=f_s$	DLM	$\frac{DLM}{LLM}$	$DL+2LL=f_s$	$DL+LL=f_s$
$.20L_1$	80.0	1328.1	.060	38900	90.4	.068	40000	20600
$.40L_1$	79.5	1864.5	.043	39200	89.9	.048	40000	21000
$.50L_1$	50.7	1792.8	.028	39400	57.3	.032	40000	20600
$.55L_1$	29.0	1762.8	.016	39700	32.7	.019	40000	20400
$.60L_1$	3.1	1667.4	.002	40000	3.5	.002	40000	20000
$.65L_1$	−28.1	1508.0	−.019	40400	−31.8	−.021	40000	19800
$.70L_1$	−62.4	1297.8	−.048	41000	−70.5	−.054	40000	19400
$.80L_1$	−148.0	564.6	−.262	47000	−167.3	−.296	40000	16400
$.80L_1$	−148.0	−2111.2	.070	38700	−167.3	.079	40000	20800
$.90L_1$	−260.4	−2819.0	.092	38300	−294.3	.104	40000	21000
$1.00L_1$	−392.9	−3950.6	.099	38200	−444.0	.112	40000	21000
$.20L_2$	−73.9	−1565.1	.047	39100	−83.5	.053	40000	20400
$.20L_2$	−73.9	564.5	−.131	43000	−83.5	−.328	40000	16000
$.30L_2$	16.0	1144.2	.014	39700	18.0	.016	40000	20200
$.50L_2$	84.4	1674.6	.050	39000	95.4	.057	40000	20500
Relative Quantities Both Concrete and Steel				100%				90%

*Live load same for all designs.

TABLE 7—LIGHT STRUCTURE, HEAVY LIVE LOAD—COMPARISON OF STRESSES AND MATERIAL QUANTITIES

Designed for *DL* & *LL*

Point	Design basis $f_c = 1200$, $f_s = 20000$				Design basis $f_c = 1200$, $f_s = 22000$			Design basis $f_c = 1200$, $f_s = 24000$		
	DLM	*LLM**	$\frac{DLM}{LLM}$	$DL+2LL=f_s$	*DLM*	$\frac{DLM}{LLM}$	$DL+2LL=f_s$	*DLM*	$\frac{DLM}{LLM}$	$DL+2LL=f_s$
$.20L_1$	80.0	1328.1	.060	38900	85.2	.064	42700	87.7	.066	46500
$.40L_1$	79.5	1864.5	.043	39200	84.4	.045	43000	87.2	.047	46900
$.50L_1$	50.7	1792.8	.028	39400	53.8	.030	43400	55.6	.031	47300
$.55L_1$	29.0	1762.8	.016	39700	30.7	.017	43600	31.7	.018	47600
$.60L_1$	3.1	1667.4	.002	40000	3.3	.002	43900	3.4	.002	47900
$.65L_1$	−28.1	1508.0	−.019	40400	−29.9	−.020	44400	−30.8	−.020	48500
$.70L_1$	−62.4	1297.8	−.048	41000	−66.2	−.051	45200	−68.4	−.053	49300
$.80L_1$	−148.0	564.7	−.262	47100	−157.2	−.278	52500	−162.3	−.288	57700
$.80L_1$	−148.0	−2111.2	.070	38700	−157.2	.074	42500	−162.3	.077	46300
$.90L_1$	−260.4	−2819.0	.092	38300	−277.4	.098	42000	−285.5	.101	45800
$1.00L_1$	−392.9	−3950.6	.099	38200	−417.1	.105	41900	−430.7	.109	45600
$.20L_2$	−73.9	−1565.1	.047	39100	−78.5	.050	42900	−81.0	.052	45400
$.20L_2$	−73.9	254.5	−.290	48100	−78.5	−.308	54000	−81.0	−.318	59200
$.30L_2$	16.0	1144.2	.014	39700	16.9	.015	43700	17.5	.015	47600
$.50L_2$	84.4	1674.6	.050	39000	89.6	.053	42800	92.5	.055	42800
Relative Quantities	Concrete 100% Steel 100%				Concrete 106.5% Steel 90.5%			Concrete 109.6% Steel 80.5%		

*Live load same for all designs.

TABLE 8—LIGHT STRUCTURE, HEAVY LIVE LOAD—COMPARISON OF STRESSES AND MATERIAL QUANTITIES

Designed for *DL* & *LL*

Point	Design basis $f_c = 1200$, $f_s = 20000$					Design basis $f_c = 1050$, $f_s = 22000$			Design basis $f_c = 1050$, $f_s = 24000$		
	DLM	*LLM*†		$\frac{DLM}{LLM}$	$DL+2LL=f_s$	*DLM*	$\frac{DLM}{LLM}$	$DL+2LL=f_s$	*DLM*	$\frac{DLM}{LLM}$	$DL+2LL=f_s$
$.20L_1$	80.0	1328.1	−497.1	.060	$^{*}38900^{43800}$	93.1	.070	42100^{49100}	95.0	.072	46400^{53700}
$.40L_1$	79.5	1864.5	−994.2	.043	39200^{41700}	92.6	.050	42900^{46200}	94.4	.051	46800^{50500}
$.50L_1$	50.7	1792.8	−1242.8	.028	39400^{40800}	59.0	.033	43300^{45300}	60.2	.034	47200^{49200}
$.55L_1$	29.0	1762.8	−1367.1	.016	39700^{40400}	33.7	.019	43500^{44600}	34.3	.020	47500^{48600}
$.60L_1$	3.1	1667.4	−1491.4	.002	40000^{40100}	3.6	.002	44000^{44000}	3.6	.002	47900^{48000}
$.65L_1$	−28.1	1508.0	−1615.7	−.019	40400^{39700}	−32.7	−.022	44500^{43600}	−33.4	−.022	48500^{47500}
$.70L_1$	−62.4	1297.8	−1740.0	−.048	41000^{39300}	−72.6	−.056	45300^{43100}	−74.0	−.057	49500^{47000}
$.80L_1$	−148.0	564.7	−2111.2	−.262	47000^{38700}	−173.4	−.306	44000^{42300}	−176.8	−.313	58900^{46200}
$.90L_1$	−260.4	−2819.0	635.0	.092	38300^{53900}	−303.2	.107	41900^{64100}	−309.2	.110	47100^{70800}
$1.00L_1$	−392.9	−3950.6	705.6	.099	38200^{65300}	−457.5	.116	41700^{84600}	−466.4	.118	45500^{94500}
$.20L_2$	−73.9	564.5	−1565.1	−.131	43000^{39100}	−86.1	−.153	48000^{42800}	−87.8	−.155	52400^{46700}
$.30L_2$	16.0	1144.2	−991.0	.014	39700^{40300}	18.6	.016	43600^{44400}	18.9	.017	48000^{48500}
$.50L_2$	84.4	1674.6	−670.0	.050	39000^{42900}	98.3	.058	42800^{47700}	100.2	.060	46600^{52200}
Relative Quantities			Concrete 100% Steel 100%				Concrete 116.4% Steel 79.3%			Concrete 118.7% Steel 70.4%	

*Stresses in top steel. **Stresses in bottom steel. †Live load same for all designs.

TABLE 9—LIGHT STRUCTURE, HEAVY LIVE LOAD—COMPARISON OF STRESSES AND MATERIAL QUANTITIES

Designed for *DL* & *LL*

Point	Design basis $f_c = 1200$, $f_s = 20000$				Design basis $f_c = 975$, $f_s = 22000$			Design basis $f_c = 975$, $f_s = 24000$		
	DLM	*LLM**	$\frac{DLM}{LLM}$	$DL+2LL=f_s$	*DLM*	$\frac{DLM}{LLM}$	$DL+2LL=f_s$	*DLM*	$\frac{DLM}{LLM}$	$DL+2LL=f_s$
$.20L_1$	80.0	1328.1	.060	38900	103.6	.078	42400	107.5	.081	46200
$.40L_1$	79.5	1864.5	.043	39200	103.0	.055	42800	106.9	.057	46700
$.50L_1$	50.7	1792.8	.028	39400	65.7	.037	43200	68.1	.038	47100
$.55L_1$	29.0	1762.8	.016	39700	37.5	.021	43500	38.9	.022	47500
$.60L_1$	3.1	1667.4	.002	40000	4.0	.002	44000	4.1	.002	47900
$.65L_1$	−28.1	1508.0	−.019	40400	−36.4	−.024	44500	−37.8	−.025	48600
$.70L_1$	−62.4	1297.8	−.048	41000	−80.8	−.062	45500	−83.9	−.064	46500
$.80L_1$	−148.0	564.7	−.262	47000	−191.7	−.339	55300	−198.9	−.352	61000
$.80L_1$	−148.0	−2111.2	.070	38700	−191.7	.091	42200	−198.9	−.092	45900
$.90L_1$	−260.4	−2819.0	.092	38300	−337.2	.119	41600	−350.0	−.124	45300
$1.00L_1$	−392.9	−3950.6	.099	38200	−508.7	.113	41500	−528.0	−.133	45200
$.20L_2$	−73.9	−1565.1	.047	39100	−95.7	.061	42700	−99.3	−.064	46600
$.20L_2$	−73.9	254.5	−.290	48100	−95.7	−.376	42200	−99.3	−.390	46100
$.30L_2$	16.0	1144.2	.014	39700	20.7	.018	43600	21.5	.019	47600
$.50L_2$	84.4	1674.6	.050	39000	109.3	.065	42800	113.4	.068	46400
Relative Quantities			Concrete 100% Steel 100%				Concrete 129.5% Steel 77.9%			Concrete 134.4% Steel 77.0%

*Live load same for all designs.

TABLE 10—LIGHT STRUCTURE HEAVY LIVE LOAD

Comparison of Concrete and Steel Quantities

	Conventional Design						Balanced Design	
Designed for $f_c = 1{,}200$ $f_s = 20{,}000$	$f_c = 1{,}200$ $f_s = 22{,}000$	$f_c = 1{,}200$ $f_s = 24{,}000$	$f_c = 1{,}050$ $f_s = 22{,}000$	$f_c = 1{,}050$ $f_s = 24{,}000$	$f_c = 975$ $f_s = 22{,}000$	$f_c = 975$ $f_s = 24{,}000$	$.5DL + LL$ $f_c = 1{,}200$ $f_s = 20{,}000$	$.5DL + LL$ $f_c = 1{,}050$ $f_s = 20{,}000$
Concrete = 100%	106.5	109.6	116.4	118.7	129.5	134.4	99	113
Steel = 100%	90.5	80.5	79.3	70.4	77.9	70.0	99	90

all of the load is transferred to the steel. Therefore in this type of design it seems logical to assume that the concrete is not taking the diagonal tension and compute the stirrups on the basis of carrying all the shear at the same permissible stress as used in the main steel.

It will be seen from the shear diagram that the different shear curves intersect at point "A." This would be so no matter what value of k were used. Therefore the factor of safety for shear is the same at this point for all values of k.

The curve marked S-60bjd shows the shear for which steel would be provided by present methods of design assuming the concrete to carry 2 per cent of f'_c. It can be seen that an inadequate amount of steel has been provided for ultimate design from the left support to about $0.5L_1$ and at the point "A" none would be provided except for code provisions which require a minimum spacing based on depth.

From this diagram it can be seen that the total amount of steel required by either design is practically the same but that balanced design probably gives a better distribution.

No shear diagram for the light structure is shown but the heavy structure would present the more unfavorable case for balanced design.

The actual bond stresses can be computed from the equation

$$u' = u \frac{DL + nLL}{kDL + LL}$$

where u = allowable bond stress as specified in codes.

This is the same equation as No. (3) for moment stress in steel except that u has been substituted for f_s and DL and LL represent shears rather than moments.

LARGE $\frac{DL}{LL}$ RATIOS

In some types of structures the $\frac{DL^*}{LL}$ ratio may be great enough so that the stress due to DL alone exceeds that which would be deemed permissible. For instance, assume that a factor of safety F_D on the dead load is desired when dead load alone is acting. Then the ratio of $\frac{DL}{LL}$ beyond which it will be necessary to design for dead load alone may be found as follows:

$$DL + F_L LL = f_{yp}, \text{ and } F_D DL = f_{yp}$$

then $$\frac{DL + F_L LL}{DL} = F_D$$

*This ratio, as stated in footnote of the introductory paragraphs, is the ratio of the stresses produced by dead load and live load respectively.

since $k = \frac{1}{F_L}$, $\frac{DL + \frac{LL}{k}}{DL} = F_D$ $\therefore$ $\frac{DL}{LL} = \frac{1}{k(F_D - 1)}$

Assuming $k = 0.5$ and $F_D = 1.5$, then where $\frac{DL}{LL} > 4$ the structure should be examined for dead load only. In such a case the design would be based on a stress of

$$f_s = k f_{vp}$$

and the load used for obtaining moment would be $1.5DLk = .75DL$.

CONCLUSIONS

That failures of reinforced concrete designed by conventional methods are relatively few considering the great amount of such construction does not necessarily prove the adequacy of the method. It does prove that the carrying capacity of the structures so designed is great enough to carry the loads to which they have been subjected. The actual factor of safety may be 1.1 or 10 and may vary widely from point to point in any structure or in any member of that structure. It may be true that loading to cause excessive stresses near the point of counter-flexure is more improbable than at other points where the factor of safety is greater; nevertheless, such loading is possible, and if it did occur the safety of the structure would depend upon the excess strength known to exist in reinforced concrete when sections are chosen by the present theories of straight line stress distribution or upon a redistribution of stress.

It is felt that the first approach to a restudy of the problem of reinforced concrete design should begin by eliminating the most conspicuous deficiencies such as have been pointed out in this study. This discussion and study have been prepared in the hope that they would hasten the adoption of a more rational and consistent method of design than now in use by removing the anomoly of having a factor of safety of about unity at some points in a member and of two, three, four or five at other places. Such a method will give a better distribution of materials by thus balancing the design; at the same time the amount of work required of the designer is not increased nor is he forced to learn new methods or construct new design aids in the form of charts and tables.

Discussion of this paper should reach the ACI Secretary, in triplicate, by April 1, 1943, for publication in the JOURNAL for June, 1943

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A Semi-Circular Arched Conduit With Uniform Symmetrical Loading*

By STANLEY U. BENSCOTER†

SYNOPSIS

The conduit is first considered to be divided into two parts, the base slab and the arched frame. The fixed end moments, fixed-end shears and stiffness value for the arched frame are presented by formulas and graphs. From these values and similar well known values for the base slab we may determine the final moments in the conduit by a single distribution of moments at one corner by the usual method of Moment Distribution. A "shear correction factor" is given to change the fixed-end shear of the frame to the final shear. The formulas and graphs take exact account of the conduit wall thickness and special considerations are given to the indeterminate state of strain in the corner region.

INTRODUCTION

The shape of conduit herein considered is shown by the line diagram in Fig. 1(a). The arched roof and sidewall are considered to be of constant thickness. The loading on top and sides is considered to be uniform as indicated in Fig. 1. This loading condition should prove adequate for the design of conduits beneath high earth fills. The distribution of the base reaction will not be discussed but is merely assumed to be symmetrical. The effects of the shortening of the perimeter of the conduit due to thrust are neglected.

The above conditions permit direct and simple integration of the integral equations of continuity which govern the moments in the continuous frame in accord with elastic theory. Fixed-end moments and shears, and stiffness, can thus be determined for the frame shown in Fig. 1(b.

Conduits beneath high fills such as earth dams must have very thick walls. This thickness sometimes exceeds 25 per cent of the span of the

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conduit. A consideration of wall thickness in writing the expression for bending moment at any point in the frame leads to a second method of analysis. Considering the sidewall and base slab each to have an infinite I in the region of the corner leads to a third method. Method 1 should be satisfactory for a preliminary analysis. Methods 2 and 3 may be regarded as upper and lower bounds on the solution, the exact solution lying somewhere in between.

It becomes convenient in the analysis and design of single barrel conduits to use a "modified bending moment" sign convention which states that positive bending moment causes maximum compressive stress in the outer fiber.

The scope of this paper has been limited to accomplish three objects: (1) to illustrate the possibility of determining and using the stiffness and fixed-end moments for a frame, (2) to present the graphs required for actual design of this shape of conduit, (3) to show a reasonable method of taking account of the effect of wall thickess in large conduits.

NOTATION

C = reciprocal of end-rotation constant
C_H = shear correction coefficient
E = modulus of elasticity
h = thickness of base slab
H = shear at corner
I = moment of inertia
K_H = shear with unit rotation (See Fig. 4)
K_M = stiffness
m = bending moment
M = bending moment at corner
$q = 1 - \left(\frac{t}{2R}\right)^2$
r = carry-over factor
R = radius of roof to center line
S_H = a shear coefficient
S_M = stiffness coefficient
t = thickness of roof and side wall
T = thrust at corner
w = uniform load (See Fig. 1)
$\alpha = \frac{h}{2R}$
β = joint rotation angle
Φ = fixed-end shear coefficient
Ψ = fixed-end moment coefficient
ϵ = ratio of side wall height to R
Θ = angle of polar coordinates

METHOD 1

Let us consider the conduit shown in Fig. 1(a) to be divided into two parts, the frame and the base slab, as shown in (b) and (c). If we could find readily values for fixed-end moment and stiffness for the frame, and for the slab, we could find the final moment at point G by a single balancing of moments according to the method of Moment Distribution. Fixed-end moments and stiffness for the base slab may be determined from graphs or formulas which are available in many publications.

The determination of fixed-end moment and stiffness values for the frame is an interesting exercise in application of the elastic theory of continuous structures. The values thus determined for fixed-end thrust, shear and moment will be as follows:

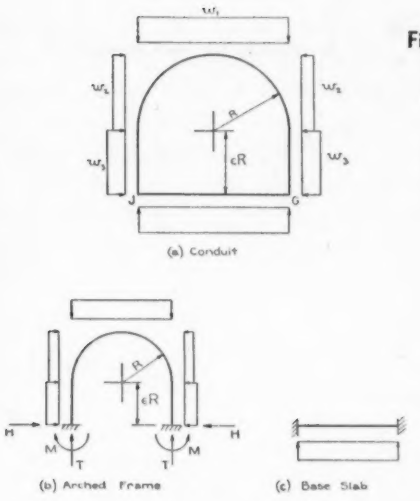


Fig. 1—Line diagram of conduit with loads

$$T = w_1 R \quad \ldots\ldots (1)$$

$$H = [\phi_1 (w_1 - w_2) - \phi_3 w_3] R \quad \ldots\ldots (2)$$

$$M = [\psi_1 (w_1 - w_2) - \psi_3 w_3] R^2 \quad \ldots\ldots (3)$$

where,

$$\phi_1 = \frac{1}{2}\left[\frac{3\pi\epsilon^2 + 16\epsilon + 2\pi}{2\epsilon^4 + 4\pi\epsilon^3 + 24\epsilon^2 + 6\pi\epsilon + 3(\pi^2 - 8)}\right] \quad \ldots\ldots (4)$$

$$\phi_3 = \frac{\epsilon}{2}\left[\frac{2\epsilon^4 + 5\pi\epsilon^3 + 40\epsilon^2 + 12\pi\epsilon + 6(\pi^2 - 8)}{2\epsilon^4 + 4\pi\epsilon^3 + 24\epsilon^2 + 6\pi\epsilon + 3(\pi^2 - 8)}\right] \quad \ldots\ldots (5)$$

$$\psi_1 = \frac{1}{4}\left[\frac{2\pi\epsilon^3 + 16\epsilon^2 + 4\pi\epsilon - 3\pi^2 + 32}{2\epsilon^4 + 4\pi\epsilon^3 + 24\epsilon^2 + 6\pi\epsilon + 3(\pi^2 - 8)}\right] \quad \ldots\ldots (6)$$

$$\psi_3 = \frac{\epsilon^2}{6}\left[\frac{\epsilon^4 + 3\pi\epsilon^3 + 30\epsilon^2 + 12\pi\epsilon + 9(\pi^2 - 8)}{2\epsilon^4 + 4\pi\epsilon^3 + 24\epsilon^2 + 6\pi\epsilon + 3(\pi^2 - 8)}\right] \quad \ldots\ldots (7)$$

A positive sign for H, T or M means that they have direction as indicated in Fig. 1(b). The fixed-end shear coefficients ϕ_1 and ϕ_3 are represented graphically in Fig. 2. The fixed-end moment coefficients ψ_1 and ψ_3 are represented graphically in Fig. 3.

The stiffness* of a beam has been defined as the moment required to produce a unit rotation at one end while the other end is held fixed. This implies that there shall be no relative translation of the ends of the beam. We may use the same definition of stiffness in the case of a frame. (See Fig. 4(a)). However, in dealing with symmetrical loads it is more convenient to evaluate a "modified stiffness" which may be defined as the

*"Continuous Frames of Reinforced Concrete," by H. Cross and N. D. Morgan, John Wiley and Sons, Inc., New York, N. Y., 1932, p. 83.

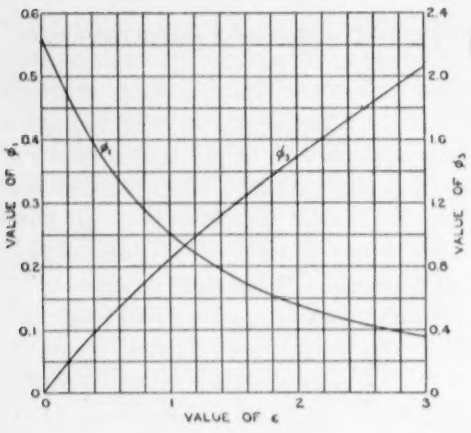
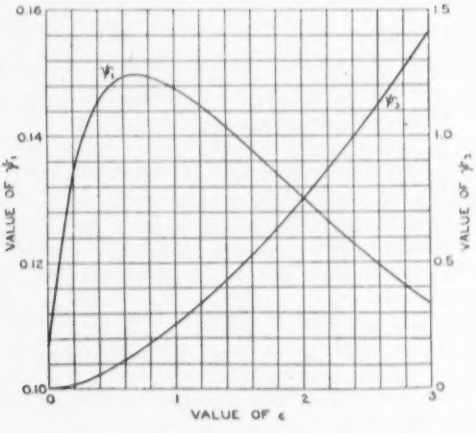


Fig. 2—Fixed-end shear coefficients

Fig. 3—Fixed-end moment coefficients



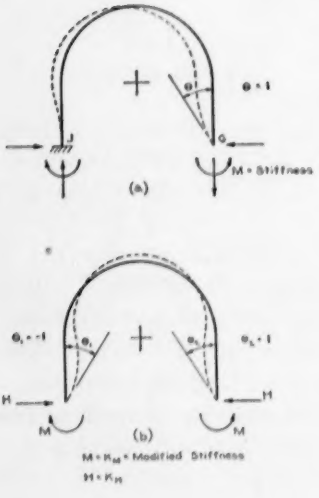


Fig. 4—Stiffness of arched frame

moment required to produce a unit rotation at one end while an equal and opposite moment is being applied at the other end. It is here implied that relative translation of the ends of the frame is prevented by the application of equal and opposite end shears, or horizontal reactions, as shown in Fig. 4(b).

The derivation of formulas for K_M and K_H as shown in Fig. 4(b) is another interesting exercise in application of the elastic theory of continuous structures. The formulas are,

$$K_H = S_H \frac{EI}{R^2} \quad \ldots\ldots (8)$$

$$K_M = S_M \frac{EI}{R} \quad \ldots\ldots (9)$$

where,

$$S_H = 12\left[\frac{\epsilon^2 + \pi\epsilon + 2}{2\epsilon^4 + 4\pi\epsilon^3 + 24\epsilon^2 + 6\pi\epsilon + 3(\pi^2 - 8)}\right] \quad \ldots\ldots (10)$$

$$S_M = 2\left[\frac{4\epsilon^3 + 6\pi\epsilon^2 + 24\epsilon + 3\pi}{2\epsilon^4 + 4\pi\epsilon^3 + 24\epsilon^2 + 6\pi\epsilon + 3(\pi^2 - 8)}\right] \quad \ldots\ldots (11)$$

The stiffness coefficient S_M is shown graphically in Fig. 7 as the curve for $\alpha = 0$.

The determination of values of fixed-end moment and stiffness for the base slab is a well known process. Such values may be obtained from graphs given in numerous publications if the slab is not of constant thickness. When these values have been determined for the frame and for the slab, we may find the final bending moment at point G by Moment Distribution. The "modified bending moment" sign convention will serve satisfactorily for this balancing process if we remember to distribute the algebraic *difference* between the fixed-end moments. Due to conditions of symmetry the final moment is obtained exactly by a single distribution. There is no carry-over to be performed.

When moments are thus distributed we say that we have allowed the joint to rotate from its fixed-end position to its final position of equilibrium. In the present problem we must release both joints simultaneously in order to preserve symmetry. We now make an interesting observation regarding end shears. Due to the simultaneous rotation of the corners there occurs no change in the end shears of the slab. But there does occur a change in the end shears of the frame which we must determine.

If the angle through which the joint rotates when it is released is $\delta\beta$ and the change in bending moment in the frame from fixed-end moment to final joint moment is δM, we may write,

$$\delta M = K_M \, \delta\beta \quad \ldots\ldots (12)$$

If δH is the accompanying change in end shear for the frame, we may write,

$$\delta H = K_H \, \delta\beta \quad (13)$$

The value of δM, which is obtained in the moment balancing process, could be substituted into eq. (12) and $\delta\beta$ thus determined. This would allow us to compute δH from eq. (13). However, the intermediate step of computing $\delta\beta$ can be eliminated if we divide eq. (13) by eq. (12) to obtain,

$$\frac{\delta H}{\delta M} = \frac{K_H}{K_M} \quad (14)$$

$$\text{or} \quad \delta H = \frac{K_H}{K_M} \quad \text{when} \quad \delta M = 1 \quad (15)$$

Thus we have determined a constant for the rate of change of thrust with respect to moment, or for the change in thrust accompanying a unit change in moment. From eqs. (8) to (11) we may write,

$$\delta H = \frac{C_H}{R}, \quad \text{when} \quad \delta M = 1 \quad (16)$$

where

$$C_H = \frac{S_H}{S_M} = 6\left[\frac{\epsilon^2 + \pi\epsilon + 2}{4\epsilon^3 + 6\pi\epsilon^2 + 24\epsilon + 3\pi}\right] \quad (17)$$

The shear correction factor C_H is shown graphically in Fig. 8 as the curve for $\alpha = 0$. We may now readily determine the change in end shear of the frame which occurs when the joint is released by multiplying the factor $\frac{C_H}{R}$ by the change in moment which occurred in the frame at the joint.

Of course, when the thrust, shear, and moment are known at point G, we may compute, by statics, the same quantities at any other section of the conduit. For example, the bending moment at any section of the first quadrant of the circular roof would be given by the following formula,

$$m = M - HR\,(\epsilon + \sin\Theta) + (w_1 - w_2)\frac{R^2 \sin^2\Theta}{2} - w_3\,\epsilon R^2\left(\frac{\epsilon}{2} + \sin\Theta\right) \quad (18)$$

METHOD 2

We shall now give consideration to the fact that the conduit wall has considerable thickness and attempt to eliminate errors involved in the process of working with a line diagram as in Method 1. If the reader will pass a section through the first quadrant of the circular roof of the

conduit as shown in Fig. 5 and write from statics a formula for bending moment, it will reduce to the following,

$$m = M - HR(\epsilon + \sin\Theta) + q(w_1 - w_2)\frac{R^2 \sin^2\Theta}{2} - w_3 \epsilon R^2\left(\frac{\epsilon}{2} + \sin\Theta\right) \quad \ldots\ldots(19)$$

where,

$$q = 1 - \left(\frac{t}{2R}\right)^2 \quad \ldots\ldots(20)$$

The only difference in the formula for moment brought about through consideration of the wall thickness is the factor q in the term containing $w_1 = w_2$. We might think of the factor q as a reduction factor to be applied to the loads w_1 and w_2. From this consideration and reference to eqs. (1), (2) and (3) we may write the following formulas for T, H, and M,

$$T = w_1\left(R + \frac{t}{2}\right) \quad \ldots\ldots(21)$$

$$H = [\phi_1 q\ (w_1 - w_2) - \phi_3 w_3]\ R \quad \ldots\ldots(22)$$

$$M = [\psi_1 q\ (w_1 - w_2) - \psi_3 w_3]\ R^2 \quad \ldots\ldots(23)$$

In the commonly occurring sizes of conduits beneath earth dams the factor q will range from .96 to .98.

An elastic analysis leading to eqs. (22) and (23) assumes the behavior of the joint G to be as indicated in Fig. 5(b). The slab is considered as a member extending horizontally to point G and the side wall is considered as a member extending downward vertically to point G. Both members are considered to occupy the shaded region. Such assumptions permit too much deflection of points G′ and G″ and probably too much rotation of the elastic curve at these points. This method of solution provides an upper bound which will not be reached by the exact solution. We may establish a lower bound upon the solution by performing an elastic analysis which assumes no displacement or rotation of the points G′ or G″ for the fixed-end condition. Thus we are lead to the next method of analysis.

METHOD 3

If we wish to perform an elastic analysis which assumes no displacement or rotation of the elastic curves at points G′ and G″, we assume the side wall and base slab each to have an infinite I within the corner region as shown in Fig. 6. We may determine the fixed-end moment and shear on plane AB from the formulas of Method 2 for fixed-end moment and shear at point G, providing we read from Figs. 2 and 3 the coefficients

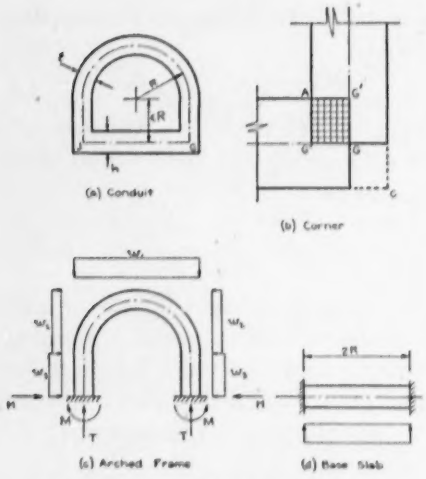


Fig. 5—Section of conduit with loads

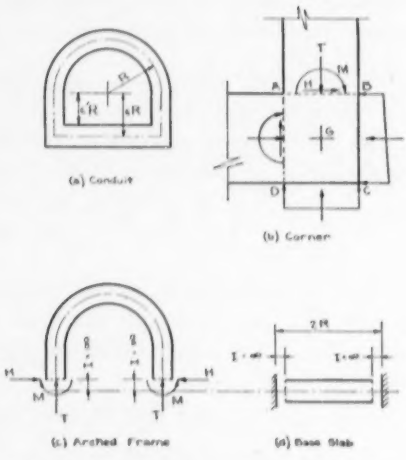


Fig. 6—Assumption of infinite rigidity in corner region

corresponding to ϵ' of Fig. 6 rather than ϵ of Fig. 5. The fixed-end moment and fixed-end shear on plane AD can readily be determined for any given assumptions of base reaction distribution.

We may imagine that we maintain the block ABCD, which is absolutely rigid, in its fixed-end position by grasping an axle which passes through the point G normal to the paper (the axle being fixed to the block). Knowing the forces which act on planes AB and BC we may compute the fixed-end bending moment on plane AC. From the forces acting on planes AD and DC we may also compute the fixed-end bending moment on plane AC. The difference between these two fixed-end moments must be distributed between the base slab and side wall in accord with their stiffnesses. The reader should note that we must not speak of the bending moment at point G but instead we must speak of the bending moment on plane AC passing through point G. When our grasp upon the axle through the point G is released the block ABCD rotates to a position of equilibrium, the final bending moment on plane AC being determined by the Moment Distribution computation. As in the previous methods, we must compute the change in end shear of the frame caused by the rotation of the joint.

An elastic analysis leads to the following formulas for the stiffness and shear correction coefficients,

$$S_M = 2\,\frac{A_1}{A_2} \qquad (24)$$

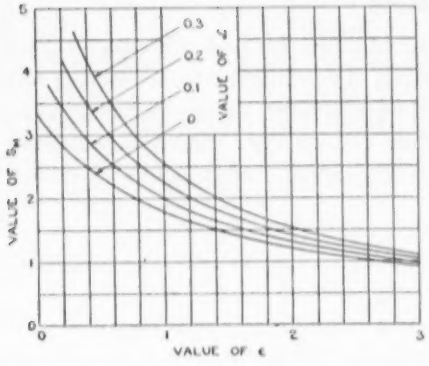
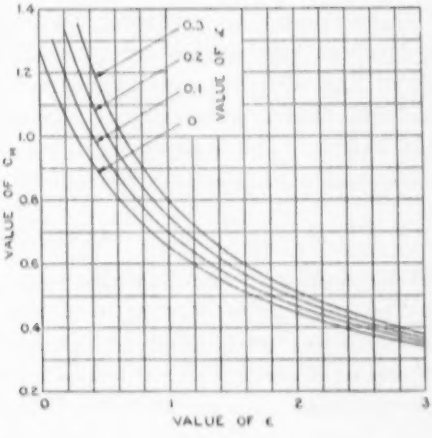


Fig. 7—Stiffness coefficient

Fig. 8—Shear correction coefficient



$$C_H = 6\,\frac{A_3}{A_1} \qquad (25)$$

where,

$$A_1 = 4\epsilon^3 + 6(\pi - \alpha)\epsilon^2 + 6(4 - \pi\alpha)\epsilon + 3\pi - 12\alpha + 2\alpha^3 \qquad (26)$$

$$A_2 = 2\epsilon^4 + 4(\pi - 2\alpha)\epsilon^3 + 12(2 - \pi\alpha + \alpha^2)\epsilon^2 + 2(3\pi - 24\alpha + 6\pi\alpha^2 - 4\alpha^3)\epsilon + 3(\pi^2 - 8) - 6\pi\alpha + 24\alpha^2 - 4\pi\alpha^3 + 2\alpha^4 \qquad (27)$$

$$A_3 = \epsilon^2 + \pi\epsilon + 2 - \alpha^2 \qquad (28)$$

These coefficients are shown graphically in Figs. 7 and 8. Having determined the bending moment on plane AC and the corrected shear on plane AB we may compute by statics the moment on plane AB and hence the moment, shear, and thrust at any section of the conduit.

BASE SLAB

The determination of fixed-end moment for the base slab to be used in Method 1 is a well known procedure. The stiffness to be used is not the usual stiffness value but a "modified stiffness" defined exactly as in the case of the arched frame. We may obtain the "modified stiffness" by multiplying the true stiffness for the slab by an end-rotation constant* given by the following formula,

$$\frac{1}{C} = 1 + r \quad \ldots\ldots (29)$$

(The carry-over factor r is negative in a bending moment sign convention.) In the case of a slab of constant thickness this end-rotation constant becomes $\frac{1}{2}$.

These comments regarding the base slab also apply in Method 2. In Method 3 we may obtain the fixed-end moment and shear on plane AD of Fig. 6(b) from existing formulas or graphs. The stiffness of the base slab, due to symmetry, is exactly equal to the stiffness of the clear span length. The reader should verify this statement by application of moment-area principles or by merely drawing the deflection curve.

CONCLUSIONS

A conduit with semi-circular arched frame and horizontal base slab may be analyzed according to elastic theory by performing a Moment Distribution computation at one corner. To this end formulas and graphs for fixed-end shear, fixed-end moment and stiffness values have been presented. A method of solution is first discussed on the basis of a line diagram. Thickness of the conduit is then taken into account and two methods of solution are developed which serve as the upper and lower bounds upon the exact solution. The latter cannot at present be determined. Only uniform, symmetrical loading has been considered.

*See previous reference, p. 119.

Discussion of this paper should reach the ACI Secretary, in triplicate, by April 1, 1943, for publication in the JOURNAL for June 1943.

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JOURNAL
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Vol. 14 No. 4 7400 SECOND BOULEVARD, DETROIT, MICHIGAN February 1943

Job Problems and Practice*

"JPP" is a means toward realizing more fully than in a limited number of longer JOURNAL contributions, the mutuality of ACI Membership effort to do a better concrete job. In JPP many Members may participate in few pages. So, if you have a question, ask it. If an answer is of likely general interest, it will be briefed here (with authorship credit unless the contributor prefers not). But don't wait for a question. If you know of a concrete problem solved—in field, laboratory, factory, or office—or if you are moved to constructive comment or criticism, obey the impulse; jot it down for JPP. Remember these pages are for informal and sometimes tentative fragments—not the "copper-riveted" conclusiveness of formal treatises. "Answers" to questions do not carry ACI authority; they represent the efforts of Members to add their bits to the sum of ACI Member knowledge of concrete "know-how."

Gap Gradings for Concrete Aggregates (39-125)

By L. BOYD MERCER†

"As always one rediscovers the laws of 'discontinuous mixtures' (i.e. jump gradings) which I proposed some fifty years ago and which is still so greatly contested"—M. Feret. This remarkable man, whose investigations upon the granulometric composition of sands dominated the early history of concrete aggregate grading, has few supporters for his theory today. What is the reason?

W. B. Fuller through his Jerome Park, New York, experiments and his curve of maximum density appears to be responsible for commencing the landslide against the theory of Feret in the first years of this century. Sanford E. Thompson who collaborated with Fuller in his tests, stated "Mr. Fuller's method of proportioning the materials so that their mixture will form a smooth, clearly defined curve appears, on its face, to conflict with Mr. Feret's conclusion that the best mixture of sand and cement

*By some unaccountable oversight identifying numbers were not given with titles of new subjects introduced in the January 1943 JPP Section. The new titles have the following numbers in A.C.I. records: From C. C. Everhart, p. 213 (39-121); Mr. Collins' contribution, p. 214, supplementing what appeared in November (39-120); by F. L. Fitzpatrick, p. 215, (39-122); R. F. Adams, p. 217, (39-123); annonymous, about rails attached to bridge decks, p. 218 (39-124); thus the February contributions are numbered (39-125, etc.).—EDITOR

†Chief Engineer, The Hydro-Electric Commission of Tasmania, Hobart, Tasmania.

for mortar is made up of coarse and fine grains only, with no intermediate grains. For sand mortars, Mr. Feret's methods are undoubtedly more exact than Mr. Fuller's, but for a concrete mixture the conditions are different. . . ." The actual features of this difference, if any, are the essential part of the problem.

Professor McNeilly carried out tests at the Vanderbilt University in 1915 and presented the following conclusions, "The best sizing of grains in a commercially-sieved aggregate is about as follows: 53 per cent to be caught between the No. 4 and No. 10 sieves; 47 per cent fines to be passed by the No. 40 sieve (this includes the cement). There is reason to believe that the coarse aggregate in concrete could be beneficially sized on a basis similar to the above; that is to say, there should be a jump in the size from coarse to quite fine instead of the usually accepted graded material."

Attention then appeared to become focussed upon the work carried out at the Structural Materials Research Laboratory in U.S.A. and the question of grading became clouded by the work of Prof. Duff A. Abrams and the Fineness Modulus. Considerable confusion became evident before it was generally realized that the fineness modulus gave no idea of the grading but was just a useful index of the coarseness or fineness of any material. Since an infinite number of gradings can be obtained that will give the same fineness modulus, and since these gradings can range from continuous to discontinuous, it is not surprising that the fineness modulus has fallen from favour as a method for designing concrete mixes.

Meanwhile the shortcomings of the Fuller parabola of maximum density had become apparent and the Swiss engineer, J. Bolomey, devised a modified curve which gave superior results primarily by providing for an increase in the percentage of fine particles. H. N. Walsh of University College, Cork, published a series of grading charts and generally speaking, the selection of aggregate grading became largely a matter of personal preference. The effect of vibration upon concrete made with various gradings next received some consideration but, it is believed, not as much as the subject warranted.

Throughout these vacillations we find isolated investigators frequently returning to the ideas of Feret. A. Caquot for instance submitte dthat the grain sizes for a coarse aggregate should fall in the order 0.4, 1.6, 6.3 and 25 mm.; a formula for size-grouping of particles to avoid interference was devised and successfully used in placing 50,000 cu. yd. of concrete in the railroad terminal at Los Angeles, Calif.; John J. Earley when entrusted with the work of providing a permanent concrete facing for the Edison Memorial Tower adopted gap gradings and stated that for this class of work "the properties of discontinuous mixtures are unique and that, all

of other things being equal, they are to be preferred over other known types of gradation for aggregates."

The fact remains that the general concensus of engineering opinion is at present in favour of continuous grading. The Bureau of Reclamation sets a standard in its *Concrete Manual* with the statement, "Experience has demonstrated that either very fine or very coarse sand, or aggregate having a large deficiency or excess of any size fraction is generally undesirable, and that material having a smooth grading curve will produce more satisfactory concrete." A very forthright statement but how far does the term "satisfactory" take us? We also find the *Manual* contains the sentence "Sand having a smooth grading curve of regular shape cannot always be obtained economically, neither is it always the most desirable." It is interesting to note that the section dealing with grading was re-written even between the first edition 1938 and the third edition 1941.

Appreciating that there must be some truth in all the contentions outlined above it is considered desirable to return to fundamental principles, examine the effect of vibration, limitations of segregation, and generally establish the point of divergence. It is contended that by so doing we bring gap gradings to the important position which they should hold in the design of concrete mixes.

Concrete Tanks for Fuel Oils (39-126)

Before the end of the present emergency many studies made and in progress will probably provide much valuable information on the use of concrete tanks for heavy and light oils. Meanwhile many inquiries are coming to the Institute. Much new information is not being disseminated these days with peace-time speed. Through the courtesy of ACI members a brief summary of information is presented, based largely on experience prior to the present war.

About 1920 the Portland Cement Association compiled a list of 394 plants having concrete tanks for the storage of petroleum oils. One hundred eighth-one replies to a questionnaire sent to these plants in 1940 showed that 102 installations were still in use; 62 had been discontinued due to plant changes; 7 had been discontinued for reasons not given, and only 10 had been discontinued because of actual difficulties with the tanks. It should be realized that all of these tanks were built prior to 1920, when the practice of concrete design and construction had not advanced to its present stage.

At the time of World War 1 considerable study was made of reinforced concrete tanks for the storage of oil. At that time it was generally

agreed that concrete tanks without any special surface treatment were satisfactory for the storage of oil heavier than 35°. For lighter oils some protection was thought desirable to prevent loss by penetration of the oil. With the better quality of concrete now in use, it seems reasonable to expect that somewhat lighter oils can be stored satisfactorily without a surface treatment.

Some of the old tanks were coated with portland cement plaster and others were painted with a mixture of portland cement and water. If carefully applied in two or three coats, these should be expected to reduce or eliminate penetration of the lighter oils.

For gasoline and some of the other very light oils, it is generally agreed that a lining or surface treatment is necessary to prevent loss from penetration. Some government agencies are recommending the use of Thiokol* or Amercoat* for gasoline storage tanks. Thiokol is applied in sheet form; Amercoat is applied as a paint. There are several other proprietory materials advocated for this purpose and it is quite probable that some of them are satisfactory but information is lacking.

Of course, the first requirement for a satisfactory job is a structural design which will prevent cracks. To be assured of this the design and construction should be under the supervision of a competent structural engineer familiar with the local conditions and requirements. Suggestions for the design of relatively small underground tanks are given in Portland Cement Association Publication, "Underground Concrete Tanks." Another P.C.A. publication dealing with the design of large tanks is "Circular Concrete Tanks without Prestressing."

Much current study is being given to the use of prestressed reinforced concrete for tanks to prevent cracking and to save steel.

Would First Mix Aggregates and Cement Dry (39-127)

By F. L. FITZPATRICK†

In the JOURNAL of November, 1941, we here have noted with interest the Report of Committee 614, Proposed Recommended Practice for Measuring, Mixing and Placing Concrete."‡ This will probably arrive too late for consideration as discussion, but I thought I might still place the following comments before you:

Part II, Section 5 places no limit on moisture content of aggregates, other than "adequate drainage."

Part III, Section 5 prescribes that "water for the batch should be released first" (i.e. before other materials commence to enter).

*The Thiokol Corp., 780 North Clinton St., Trenton, N. J.; and (Amercoat) The American Pipe & Construction Co., Box 3428, Terminal Annex P. O., Los Angeles, Calif.

†General manager, Rocla Limited, concrete engineers, Melbourne, Australia.

‡Since adopted as an ACI Standard (ACI 614-42).

TABLE 1—TENSILE STRENGTH TESTS ON STANDARD BRIQUETTES

Sample	Mixing Procedure	Average Tensile Strength		
		1 Day Damp Air 6 Days Water	1 Day Damp Air 27 Days Water	1 Day Damp Air 6 D. Water 21 D. Air
(a) 300 Parts Standard Sand 100 Parts Cement 40 Parts Water	Water added after cement and sand well mixed dry	354	385	485
(b) Same as above	Water + cement made to a slurry and then added to sand	220	280	315

Part III, Section 13 provides for "shading and sprinkling" aggregates in hot weather.

In the Research Laboratory of my company, we have made extensive experiments with cement-sand mixes, which consistently indicate that initial moisture content of sand is important and that sand used in a saturated state, even though moisture content is allowed for in gauging water, produces a serious drop in tensile strength. We are investigating whether a similar result applies also to coarse aggregate.

We were led to investigate this subject because we found, when using pozzolanic admixtures, that wet grinding of the pozzolana and adding the pozzolana slurry to the mix, gave notably lower strengths than when added as a dry powder, total water and all other factors remaining constant.

We first tried the effect, upon tensile strengths, of pre-mixing the cement and water, and then adding aggregates. The strength fell (Table 1).

Next we tried the effect of different moisture contents in the sand. Typical results were as per Table 2. Repeat tests have always confirmed the indications shown by the tests cited.

Our present belief is that when the initial water content of a sand is such that an appreciable water film surrounds each particle, the cement grains fail to make such close contact with the sand particles as when dry or just damp sand is first mixed and the cement and then the water, added.

In our tests, we used a normal portand cement, and "standard" sand. Compaction was by hammer machine, and specimens were tested on standard Avery Briquette Testing (tension machine).

TABLE 2—TESTS OF TENSILE STRENGTH

Mix: 300 Parts by weight Standard Sand (Dry weight)
100 Parts by weight Normal Portland Cement
40 Parts by weight Water (total).

Series No.	Initial Moisture Content of Sand	Water Added and Order of Mixing	Average Tensile Strength After Storage: 1 Day Damp Air 27 Days Water
1	Nil	40 Parts added to pre-mixed sand and cement	345 psi.
2	8% of sand weight = 24 Parts	100 Parts cement mixed with the damp sand then 16 Parts water added	353 psi.
3	13.3% of sand weight = 40 Parts	100 Parts cement mixed with the wet sand. No further water added.	328 psi.

NOTE: (a) The sand in Series No. 2 was in a "damp" but not "wet" state.
(b) The sand in series No. 3 was in a "wet" state, similar to the state of sand piles in the open for some time after rain has fallen.

We believe that it is important for first class concrete, that fine aggregates, (and probably coarse also), should have a limiting moisture content when introduced to the mixer, and that water should preferably not be added to the mix until the aggregates and cement have first been well combined. For purposes of preventing clogging of mixer blades, we do not object to such an amount of water being just put in the mixer as will not exceed (together with moisture existing in aggregate) 3 per cent by dry weight of the total batch.

We commend to the attention of other concrete users and investigators a study of the effect of initial water in aggregates. We have not seen any reports of research into this matter, which seems of some importance.

If our view is correct, not only does strength drop, but density must be impaired, and durability affected, by using aggregates in a saturated condition, or by allowing water and cement to mix first, or even by too much water being in the mixer prior to entry of the solids.

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Current Reviews

of Significant Contributions in Foreign and Domestic Publications, prepared by the Institute's Reviewers

Finishes for portland cement structures

H. A. GARDNER, National Paint, Varnish and Lacquer Association Circular No. 612, 1941, 43-8; Brit. Paint Res., 1941, (82), 225. *Building Science Abstracts*, XV, (New Series), No. 2, February, 1942.
HIGHWAY RESEARCH ABSTRACTS

Of some 20 paint systems, including oil and phenolic resin varnishes and paints, chlorinated rubber paints and alkyd resin emulsions, applied to concrete and exposed for 6 to 12 months, a rating of "excellent" was applied to phenolic resin spar varnish, clear or pigmented with aluminum, used in two coats or as a primer for oil paints, and to a flat wall paint used as primer for linseed oil house paint.

Determining proportions of concrete for desired compressive strengths

A. T. GOLDBECK and J. E. GRAY, *Engineering News-Record*, V. 129, No. 19 (Nov. 5, 1942), pp. 86-87.
Reviewed by S. J. CHAMBERLIN

Concrete mixtures are designed by the use of three tables: (1) dry, rodded volume of coarse aggregate per unit volume of concrete $\frac{b}{(b_o)}$; (2) cement and water contents for required strengths; (3) slumps, maximum size of aggregate and strengths for different structures and exposures. Use of the tables and the general design procedure are outlined by solving a sample problem.

Curved box girders for highway overpass

Engineering News-Record, V. 129, No. 21 (Nov. 19, 1942), pp. 62-63. Reviewed by S. J. CHAMBERLIN

Box girders of reinforced concrete were built on horizontal and vertical curves, and tilted to provide varying superelevation. The deck structure consists of four lines of girders spaced ten ft. apart, which with top and bottom slabs, form three box girders. Spans vary from 40 to 70 ft., but a constant girder depth of 4 ft. is maintained throughout. Curved girders conform to modern alignment; they are attractive; torsional moments can be easily provided for; modern methods make possible comparatively easy construction.

Roads under war restrictions—I, II & III

Engineering News-Record, V. 129, No. 15 (Oct. 8, 1942), pp. 74-78, No. 17 (Oct. 22, 1942), pp. 89-91, No. 19 (No. 5, 1942), pp. 75-78. Reviewed by S. J. CHAMBERLIN

A review of war problems in midwestern and western state highway departments and how they are being met with restricted materials, revenues and personnel. New construction is limited to highways essential to the war and is of unreinforced concrete.

Heavy trucking has required repairing and widening of old pavements. Small bridges are of timber and small culverts are of unreinforced concrete pipe, clay pipe and timber. Larger culverts are of unreinforced concrete construction. Salvaged structural and reinforcing steel is being used.

Grouting checks foundation vibration

Engineering News-Record, V. 129, No. 17 (Oct. 22, 1942), pp. 95-96. Reviewed by S. J. CHAMBERLIN

Vibrations set up by the compressors of a gas station caused serious trouble in the pile foundations of both the machines and the building in which they were housed. The vibrations were practically stopped by grouting the 50 ft. of sand through which the piles were driven. Application of grout under pressure was through solid wall pipes, perforated after driving by a gun that shot bullets through the pipe at desired locations. The neat cement grout was supplied continuously and the excess returned to prevent plugging. Maximum allowable pressure was limited in most instances to 150 psi.

Concrete jackets make up for unequal pile lengths

Engineering News-Record, V. 129, No. 17 (Oct. 22, 1942), pp. 78-81. Reviewed by S. J. CHAMBERLIN

Difficult driving was simplified by precast jackets that relieved exact requirements on length and position of reinforced-concrete piles in piers designed for loads of 1,000 lb. per sq. ft. A 4 x 4 in. timber was cut of a length such that when bolted to the inside of a jacket it would slip down over the 18-in. square pile and be supported by the timber at just the prescribed height. A timber frame was then bolted around the pile at the lower end of the jacket to support it when the 4 x 4 was removed and to serve as the bottom form for retaining the concrete which filled the jacket. Precast concrete jackets also were put around driven batter piles of untreated timber and grouted.

A.S.T.M. standards on cement

6th Edition, Am. Soc. for Testing Materials, 260 South Broad St., Philadelphia, Pa., 125 p., $1.35. ASTM REVIEW

The sixth edition of this compilation issued annually by the American Society for Testing Materials gives in compact form all of the latest standard and tentative specifications, methods of chemical analysis, and methods of physical tests pertaining to cement. Also included is information on analytical balances and weights, and the Emergency Alternate Specifications for Portland Cement (EA — C150) issued in the interest of expediting procurement. Included for the first time are the Tentative Specifications for Treated Portland Cement for Concrete Pavements (C 175—42T). A Manual of Cement Testing and a List of Selected References for Portland Cement are also given.

P.R.C. pre-stressing reinforced concrete, Part I, General Data

KURT BILLIG. 54 pp., paper bound, 5½ x 8½. Kurt Billig, New York, $3.00. Reviewed by A. J. BOASE

This booklet is a reprint of a paper presented to the Institution of Civil Engineers, Ireland, December 1, 1941. It is published in that Society's Transactions, Volume LXVIII, pages 33-90.

In general, the paper is a popular discussion of the theory of pre-stressing. It contains a short history, followed by a word picture of the methods of Freyssinet, Billig, Dischinger and Lossier. Following this are given examples of a few applications to actual structures. The author then presents his conclusions, with a short bibliography. The paper is concluded with 16 pages of discussion and communications with the author by various members of the Irish society.

Further phase-equilibrium studies involving the potash compounds of portland cement

WILLIAM C. TAYLOR, *Journal Research*, National Bureau of Standards, 29, 437, Dec. 1942, RP1512.
AUTHOR'S ABSTRACT

The present research on the system $K_2O.23CaO.12SiO_2$-CaO-$5CaO.3Al_2O_3$ is a part of the program designed to determine the manner in which the K_2O affects the compound composition of portland cement clinker. Thermal and optical data are presented and a phase-equilibrium diagram of this system has been constructed. The only compounds that were observed in the system under equilibrium conditions are the components of the system and $3CaO.Al_2O_3$. The effect of K_2O on the compound composition of clinker containing K_2O, CaO, MgO, Al_2O_3, Fe_2O_3, SiO_2, and SO_3 has been studied also. The only compounds of K_2O found to be stable are K_2SO_4 and $K_2O.23CaO.12SiO_2$, the K_2O combining preferentially with the SO_3 present. Crystals of K_2SO_4 were identified in several commercial portland cement clinkers.

Wood piles with mortar coating save time and critical materials

Engineering News-Record, V. 129, No. 19 (Nov. 5, 1942), pp. 72-74. Reviewed by S. J. CHAMBERLIN

Strong, rigid and durable piles were obtained by putting a sand-cement protective coating around long, untreated wood piling. Stipulations established by experimental studies included: minimum tip diameter of piling should be 6 in.; 4 x 4 x 1-in. notches in the wood surface should be made at intervals of 2 ft.; the wire mesh reinforcing of No. 12 or No. 14 wire should be 2 x 2-in. mesh held out ¾ in. from the wood surface at all points. The top and bottom 6-in. of that portion of the pile having the protective coating should be wrapped with 6 turns of No. 9 galvanized wire over the mesh. The 1½-in. coating of dry mixture was applied with compressed air on to the wetted piling and wet cured for three days. Damage to the coatings as the result of driving occurred on only a few piles and was ascribed to improper treatment. Small cracks developed during curing were not deemed detrimental.

Thermal characteristics of high-temperature oil well cements

E. E. BYRD and F. W. JESSEN. *Industrial & Engineering Chemistry*, Industrial Edition, Oct., 1942, Vol. 34, No. 10, pp. 1133-1252. Reviewed by ROY N. YOUNG

Five high-temperature oil well cements were studied with reference to their more important properties, including heat of hydration, rate of heat evolution, pumpability, setting time and strength. With the exception of strength these properties were examined at three different temperatures—150 F., 175 F. and 200 F. Generally the total heat evolved up to time of final set increased with increase of temperature. The maximum rate of heat evolution for each cement was increased with increase in temperature, and the time at which the maximum rate occurred was decreased. The degree to which the several cements were affected varied widely. The periods through which the cement slurries remained pumpable were not consistently related with the other physical properties investigated or with chemical composition. Three of the cements contained admixtures. These three remained pumpable longer than the others even though two of them evolved more total heat and had higher rates of heat evolution.

A novel type of dowel for concrete roads

K. KRAH: *Strassenbau*, 1941, 32 (5), 77-9. *Road Abstracts*, Vol. IX, No. 10, Oct. 13, 1942.
HIGHWAY RESEARCH ABSTRACTS

To economize in steel, and to ensure increased accuracy in the alignment of dowels, a short cylindrical porcelain dowel, 2¾ in. in diameter with hemispherical ends, has been developed in Germany to replace the normal type which consists of a 28-in. long round steel bar with wire supports. The cylindrical part of the dowel is at least as long as the maximum width of the joint. Each dowel is divided longitudinally in a vertical plane at right angles to the expansion joint: an anchor bar, resting on the foundation, is

attached to opposite ends of the half-dowels. On contraction or expansion of the slabs the two halves slide on each other, while the cylindrical body prevents vertical movement of the slabs under traffic stresses. The diameter of the dowels and the compressive strength of the material (about 32 tons per sq. in.) insure that stresses are satisfactorily transmitted. The results obtained from the experimental use of dowels of this type in a section of motor road in 1939, and consideration of the thermal stresses caused in the slabs by hot weather with particular reference to the work of K. Eberle lead to the conclusion that the new type of dowel can be substituted for the round steel bar, and is able to eliminate the formation of cracks caused by high thermal stresses.

Special section on war buildings of 1942

Engineering News-Record, V. 129, No. 17 (Oct. 22, 1942), pp. 99-128. Reviewed by S. J. CHAMBERLIN

A section devoted to description of how war buildings were built with greatly reduced familiar materials. Included are redesigns in concrete, combinations of other materials with concrete and construction methods for speed and efficiency.

A large shop building was constructed of concrete columns and laminated wood girders up to 62-ft. spans. Concrete brackets and wooden knee braces connected the girders and columns.

One portion is devoted to the planning of the War Department's office building, the world's largest building.

Walter H. Wheeler describes how an auditorium building was redesigned in concrete. T-beams of 100-ft. span were substituted for trusses, bar joists for steel purlins. One steel girder was replaced by a concrete girder 10 ft. deep and 18 in. wide.

A war-inspired innovation was a single-story concrete factory. Construction operations were featured by a wheelless traveler, tower-pavers and the use of high-early-strength concrete.

Light-reflecting concrete for factory floors

M. A. BERNS, *Civil Engineering*, Vol. 12, No. 11, Oct. 1942, p. 556-558. Reviewed by J. R. SHANK

Light reflecting floors of white cement concrete were installed in several airplane manufacturing plant buildings among which was one for the Consolidated Aircraft Corporation. White portland cement concrete 1:1:2 mix with 5 gal. of water per sack was placed as a topping on gray concrete construction. After thorough curing and cleaning the surface was sealed with a colorless, non-staining material. White or light-colored sand and stone were used as aggregate.

A survey made by the General Electric Co. when the floor was new showed that the white cement floor reflected 61 per cent more light than the adjoining gray floor and the increase in reflection increased the illumination on the vertical plane 20.6 per cent. After six months of use the white cement floor reflected 70 per cent more light than the gray floor. The standard procedure for keeping the floor clean was to sweep daily, mop once a month and scrub every two or three months.

To obtain the same vertical illumination over the gray floor, keeping in mind the importance of illumination up under wing surfaces would require, it is estimated a total outlay of $72,760. The extra cost of the white floor was estimated at $40,000. Annual savings after the first year are estimated to be $25,000 for power and lamp maintenance.

Ten year tests of high early strength cement concretes

LOUIS SCHUMAN, *Journal Research*, National Bureau of Standards, 29, 397, Dec., 1942, RP1508.
AUTHOR'S ABSTRACT

A study is reported of ten-year compressive-strength tests on concretes made with 12 high-early-strength cements, for which the results up to one year were given in *J.*

Research NBS 14, 723 (June 1935) RP 799. The concretes were of three different cement-water ratios, five initial temperature conditions, and four curing conditions.

Concretes stored in damp air generally continued to gain strength up to 10 years. Concretes stored in the air of the laboratory had about the same strengths at 10 years as at 28 days. The strengths at ages after 28 days were not appreciably affected by the initial temperatures, which varied from 70 to 110 F. However, the usual variation in strength with C/W ratio persisted up to 10 years. For damp-cured 1:2:4 concrete of C/W ratio = 1.50 (7.5 gal. of water per 94 lb. bag of cement) the compressive strengths at 10 years were generally between 4,000 and 5,000 psi. For concrete of the same proportions, and a C/W ratio of 1.73 (6.5 gal. per bag), strengths of over 6,000 psi. were attained for 8 of the 12 cements.

Cements made in 1941, even of the moderate-heat type, are shown to be capable of giving concrete strengths in 1 month at least equal to those for a 1910 cement at 10 years; present-day (1941) high-early-strength cements may give strengths at 1 month exceeding the 10-year strengths reported in Research Paper RP1508 for the 1930 cements.

Big airbase paved in ninety days

Engineering News-Record, V. 129, No. 15 (Oct. 8, 1942), pp. 69-71 Reviewed by S. J. Chamberlin

Biggest job in building a large airbase was construction of 604,000 sq. yd. of concrete and 330,000 sq. yd. of asphaltic pavement in 90 days. Placing of the concrete averaged 14,000 sq. yd. daily for 30 consecutive days and the maximum for any one day was 16,300 sq. yd. The greater length of the three 5,550-ft. runways consisted of a 150-ft. wide non-reinforced-concrete central strip built in 12½-ft. wide sections of 10-8-10-in. thickness over a 3-6-3-in. crushed rock sub-base. For 75 ft. on each side of the concrete, asphalt paving having a 9-in. macadam sub-base, a 3-in. binder, and a 1-in. asphalt topping was used. A second portion was of similar design but a lime-rock asphaltic construction consisting of an 11-in. base, a 4-in. binder and a 1-in. topping was substituted for the wide concrete middle strip. For the third design the central part was an all sand-rock asphalt, consisting of an 11-in. macadam base, a 4½-in. binder and a ½-in. topping. Bituminous shoulders were built with a macadam sub-base varying in thickness from 12 in. at the inner edge to 9 in. at the outer with a 3½-in. binder and a 1½-in. topping. Comparisons will be made to determine how the different designs stand up under similar load conditions. Concrete was placed in alternate 25-ft. wide lanes for the full length of the runway, where possible, using four pavers. The concrete was batched at a central plant. Several curing methods were tried, but the most satisfactory was bituminous-treated paper placed on the slab immediately after the finishing and kept in place from four to five days. Four by four-in. cross timbers held the paper in place.

Bond strength of reinforcing bars embedded horizontally in concrete

Rowland Cuthbert Robin, Paul Engberg Olsen, R. F. Kinnane, The *Journal* of the Institute of Engineers Australia, Vol. 14, No. 9 (Sept. 1942) pp. 201-217. Reviewed by J. R. Shank

The authors report a broad but preliminary series of tests on the bond resistance of bars cast horizontally in concrete. The result of these tests showed clearly that the higher above the bottom of the form a horizontal bar is cast, the lower is the bond strength, and the more erratic the results. A rather startling indication from pull-out tests is that bars held rigidly during pouring 7½ in. above the bottom gave average bond strengths in some instances lower than the commonly used safe working stresses. After examination of the specimens it was noted that no bond resistance was had from the lower half of the contact. In one series the first slip of bars 7½ in. up came at loads only 35 per cent of those at 1½ in. It was found that much greater bond resistance can develop if the bars are not supported from the forms and may ride on the green concrete.

A comparison of bars supported on the forms, floating on the green concrete, and cast vertically gave averages 193, 366, and 384 psi. respectively for 4000 psi concrete.

In beam tests the results were not so startling. Averages for bars cast in the tops of beams, bottoms of beams and pull-out tests showed at beam failure and ultimate pull-out, 194, 284 and 344 psi respectively. Top bars, hooked at the end and supported during pouring showed under negative bending that the deflection was approximately twice as great as that of straight bars left to float during pouring, indicating that bond or anchorage occurred only at the hooks in the former. In none of these tests were slumps excessive.

The effect of making the molds completely water-tight was to decrease the bond strength of bars cast near the bottom and to increase, somewhat,that of bars cast near the tip of pull-out blocks and beams.

Retaining walls

R. R. MINIKIN: *Engineering*, 1942, 153 (3967), 61-3. *Building Science Abstracts*, V. 15, (New Series), No. 5, May, 1942, p. 73. HIGHWAY RESEARCH ABSTRACTS

Data are given in support of the claim that the revised wedge theory is more accurate than Rankine's earth pressure theory when applied to the design of retaining walls. It is recommended that the effective pressure base of a retaining wall should be normal to the direction of the resultant thrust so that stress is normally and uniformly transmitted over the supporting soil. The cross section of wall, fill and toe should always be analyzed for shearing stress stability over the surface of some critical arc passing below the base of the wall. The resistance of the ground near the toe should be evaluated as a counter-active pressure only, and full allowance made also for reduction of weight or of shearing resistance per unit area when such ground is saturated or subjected to seepage. In all cases, exhaustive tests should be made for friction angle, cohesion, moisture content and consolidation behavior of the sub-soil. Clays of the indurated silt type have low resistance to sliding and the use of sheet-piling layouts as self-supporting walls is mentioned. Failures of retaining walls are generally due to sliding on the base or shearing of the sub-soil below the base, and only rarely to overturning or rotation about the toe. In weak ground where toe pressure is high, the intensity of the shearing stress in the sub-soil is greater than the shear resistance of the material so that, with a low safety factor for shear rupture of the sub-soil may cause failure of the wall. Two examples of failures in retaining walls are described, and analyzed graphically by the author's interpretation of the wedge theory. In the first case it proved that the system was stable in the dry (safety factor 1.53) but likely to rupture in the wet (safety factor 0.95); in the second case a wall, reinforced at the toe by large cast in situ concrete slabs, failed owing to shearing of the sub-soil accelerated by infiltration of water into the fill.

Twin drydocks constructed under water

Engineering News-Record, V. 129, No. 21 (Nov. 19, 1942), pp. 66-73. Reviewed by S. J. CHAMBERLIN

Excavation was done by three dipper, one suction and two clamshell dredges. A minimum of 2 ft. of crushed stone was placed over the floor after it has been leveled with an I-beam drag. H-piles, 12 x 12-in. and 74 lb., spaced generally 5 x 7 ft. apart, were driven to depths to top of pile of 65 ft. below water to carefully checked positions. Prefabricated box truss forms were placed under water to divide the floor sections into reasonable size pours and to confine the concrete, as well as to provide a means for holding the heavy reinforcement in place. The steel of the truss served as part of the reinforcing and was supplemented by steel shapes and reinforcing bars. The structural frame was enclosed by steel sheeting on the sides and end. Each box was 14 ft. wide, 20 ft. high and as long as the full width of the dock and walls. A frame at the wall line separated the section under the walls from that under what will be the open dock.

Forms were used for alternate 14-ft. wide sections only, the concrete being placed later in the unformed areas. Concrete was mixed by double-drum pavers and pumped 1100 ft. to scows equipped with sliding tremie pipes. Concreting was started at one end and the flow of concrete down successive tremie pipes were not started until the flow along the floor had surrounded their bottoms. Any mud on the bottom was pushed ahead of the wave of concrete to the end of the form where it was ejected by an air-lift. Wall concrete was placed by similar methods. As the concrete rose the pipes were lifted to the top of their run and a 20-ft. section of pipe removed without breaking the seal at the bottom. The 2½-ft. finished floor was placed by pumping concrete to the edge of the drydock and dropping it through elephant-trunk spouts to hoppers on the floor for final distribution by buggy. Forms for the 2-ft. thick veneer on the walls were lacquered plywood panels 14 ft. wide and the full height of the dock. Concrete for the job required 7 bags of cement per yard and was made with one part cement, 1.7 parts of sand, 2 parts 1-in. gravel, and 0.45 part of 2-in. gravel.

Stresses in the corner region of concrete pavements

M. G. SPANGLER (Research Assoc. Prof. C. E., Iowa State College) Bulletin 157, Iowa Engineering Experiment Station. 96 pages. Reviewed by IOWA STATE COLLEGE

Discusses other research studies on concrete pavement stresses, including the Bates road tests, the Westergaard analyses, and the Arlington tests by the U. S. Public Roads Administration.

In the work at Iowa Engineering Experiment Station, observations were made of the strains in the top surface of experimental concrete pavement slabs made in the laboratory; measurements made of the pressure between the slabs and the subgrade, and of the deflection of the slabs, enabling calculation of the modulus of subgrade reaction.

After the measurements were completed the slab corners were loaded to failure; observations made of the size and shape of broken corners of actual pavements, and together with the laboratory strain measurements, and the results of the Arlington tests conducted by the U. S. Public Roads Administration, led to the development of the working formula for corner load stresses.

Professor Spangler concluded that the locus of maximum stress in the corner region of a concrete slab when loaded at the corner is a curved line which bends toward the corner as it approaches the edges of the slab, and that maximum stress is not distributed uniformly along this line but is greater at the bisector and smaller at the edges of the slab. He observed that the magnitude of stress in the corner region is dependent upon the position of the center of gravity of the corner load. The modulus of subgrade reaction is not a single-valued constant for all points in the corner region but is considerably greater at the corner than at points back in the slab.

Professor Spangler made an informal survey and study of the service behavior of two sections of concrete pavement near Ames, Iowa; made an analysis of the corner breakage of these two sections, and calculated the thickness of pavement which would have been required to carry heavy traffic for a long time without excessive corner breakage.

Single copies of this 96-page bulletin may be obtained without charge from the Iowa Engineering Experiment Station, Iowa State College, Ames, Iowa.

Second Annual

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FOREWORD

A year ago the Institute inaugurated this section of the JOURNAL for advertising—*advertising with the special purpose of supplying information of value to ACI's specialized audience.* This year the Second TPS sees an increase in the number of advertisers and in the space used.

With appropriate allowances for the War's conversion of normal commercial and engineering aims, the "special purpose" of TPS seems this year to be more clearly in the minds of the participants than at the outset of the plan—a more general recognition of the fact that ACI Engineers are well informed in their special field.

Obviously, any commercial organization is inclined to be partial to the materials, products, services it has to sell. In a highly specialized technical field the buyer is, on the other hand, inclined to be partial to competent data objectively presented. In these pages ACI members hope to see these viewpoints harmonized for mutual understanding. From such an understanding will come a more complete appreciation of the considerable contribution to engineering progress made by commercial producers of the materials, tools and special skills of the concrete field.

Institute endorsement of the pages which follow in this section is neither given nor implied.

BLAW-KNOX DIVISION of Blaw-Knox Company

Farmers Bank Bldg. • Pittsburgh, Pa.—New York, Chicago, Birmingham, Philadelphia

TRUKMIXER LOADING PLANTS

Blaw - Knox Trukmixer Loading Plants include complete equipment for overhead storage of aggregates and bulk cement; also accurate weighing batchers (either manual or automatic) for measurement of aggregates, bulk cement, water, and admixtures.

Many of these plants are made up of an assembly of standard bin and batching units. Blaw-Knox also designs special plants to fit special requirements and conditions. Accuracy of proportioning; speed, economy, and ease of operation, sturdiness and durability of equipment are stressed in the design of Blaw-Knox Trukmixer Loading Plants.

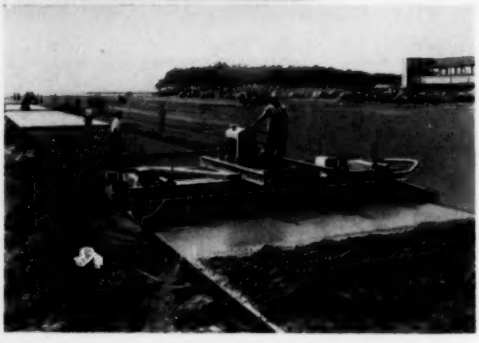
BULK CEMENT PLANTS

Complete Blaw-Knox Bulk Cement Plants include equipment for unloading cement from railroad cars or trucks, elevators for elevating cement into storage bins, the overhead storage bin, and weighing batcher equipment for measurement of cement.

They are available in a portable design which provides for easy and economical erection, dismantling, and moving. Blaw-Knox also furnishes larger bulk cement plants where large overhead storage is required, or for several brands of cement.

The weighing batcher equipment is either of the manual or automatic type, designed for easy operation and extreme accuracy. The storage bin equipment is sealed for protection of the cement against weather.

ROAD FINISHING MACHINES



The Blaw-Knox two-screed finisher for concrete paving for roads and airports is arranged for utmost flexibility in operation to take care of varying textures of concrete. The machine is equipped with six traction speeds, forward and reverse, and four screed speeds. Screeds are adjustable for crown; also, screeds with quick crown change device for flattening crown on curves for road paving are available.

Vibratory attachments for concrete paving are also furnished when specified with Blaw-Knox Finishing Machine. Accessories are available for use of the Blaw-Knox Finisher on high type asphalt paving construction.

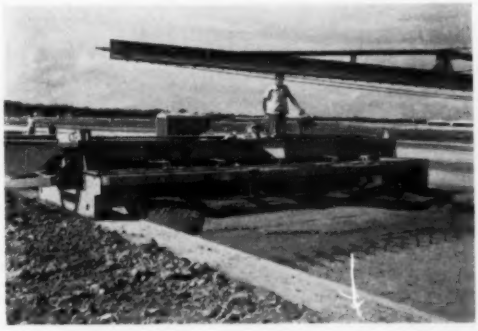
Five ft. width adjustment available; 10 to 15 ft. and 20 to 25 ft. machines are standard.

• BLAW-KNOX •

CONCRETE SPREADERS

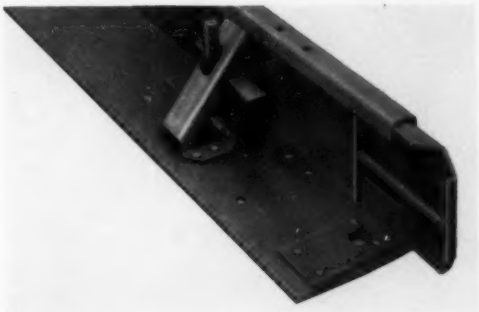
The Blaw-Knox paving Spreader has had a wide application to concrete paving on roads and airports. It is automatic in its action regardless of where the concrete is placed between the side forms. It spreads concrete for the base course, ready for the placing of the mesh reinforcing and then spreads the upper course of the concrete over the mesh to finish grade ready for the finishing machine. The spreading blade and final strikeoff are quickly and easily adjustable for elevation. The Blaw-Knox Concrete Paving Spreader easily maintains pace with the largest paving mixers in use.

Vibratory attachments for concrete paving are also furnished, when specified, with the Blaw-Knox Concrete Paving Spreader. Five ft. width adjustment available; 10 to 15 ft. and 20 to 25 ft. machines are standard.



BLAW-KNOX SELF-ALIGNING PAVING FORMS

The self-aligning feature of Blaw-Knox Paving Forms permits rapid form setting; always true to line and grade, regardless of whether the steel stakes are bent or forced off plumb in stony ground. Forms are reinforced and stiffened to carry heavy machines without distortion or appreciable deflection. Use of high carbon steel plate adds to inherent rigidity of form design.



CONCRETE BUCKETS

The Blaw-Knox Roller Gate Concrete Bucket is advantageous for the handling of mixed concrete of any consistency. The discharge gate consists of a flexible rubber pad mounted on anti-friction rollers and does not leak grout. It is tight. This design of gate can easily be closed before discharge of the complete load from the bucket. The discharge opening at the bottom of the bucket is large, and the hopper slopes are steep insuring rapid and complete discharge of dry concrete. These buckets are available in sizes ranging from 1 yard to 8 cubic yard.

The Blaw-Knox Clam Gate Concrete Bucket is a low cost unit for handling concrete of normal consistency. It is furnished in the following sizes: ½, ¾ and 1 Cubic Yard capacities.

Type R—Roller Gate

Type C—Clam Gate

THE KELLEY METHOD DELIVERS A SURFACE—CREATES A SAVING IN

The Kelley Compactor Power Float at work on dry topping thirty minutes after placing. The weight of the machine, plus the action of the compactor hammers and the rotating of the disc, compacts and floats out the topping to a smooth, level surface, bringing just enough mortar to the surface for the troweling operation.

LEADING AUTHORITIES RECOMMEND AND SAY THAT:

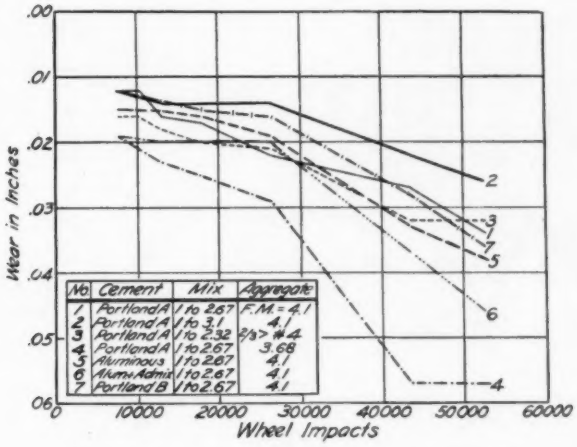
Low water-cement ratio mixes using graded aggregate give high strength, water tightness —resistance to wear—prevent segregation and allow leaner mixes.

Lean mixes of low water content have less shrinkage than rich mixes of high water content.

A concrete wearing surface using this mix has the coarse aggregate in the surface to take the wear; eliminates dusting caused by excess water bringing fines to surface and eliminates cracking and crazing caused by water shrinkage and excess fines.

KELLEY ELECTRIC MACHINE COMPANY

SUPERIOR WEARING CONCRETE LABOR AND CEMENT COSTS



Wear of Seven Sample Segments of Test Floor

Reference: Journal of American Concrete Institute, Nov.-Dec. 1934 & Sept. 1938

MIX No. 2—Leanest in cement content—best wearing qualities.

MIX No. 4—Richer in cement content and 12% more fines passing 50 mesh sieve—poorest wearing qualities.

The coarse aggregate at surface takes the wear.

Excess fines and cement lower wearing qualities.

The Kelley Compactor Power Float provides a mechanical means for working the recommended, harsh, dry mixes and delivers a dense, durable, level, dustless hard-wearing surface at a saving in labor and cement costs. The rotating disc moves the material and delivers an even, floated surface. At the same time, the weight of the machine, plus the 1200 per minute hammer blows break down the bridging of the aggregate, drive out impacted air and densify the mix throughout its thickness without segregation, leaving no room for shrinkage—the cause of cracking.

The Kelley Compactor Power Float is built in both Gasoline and Electric Models for all types of concrete wearing surfaces:

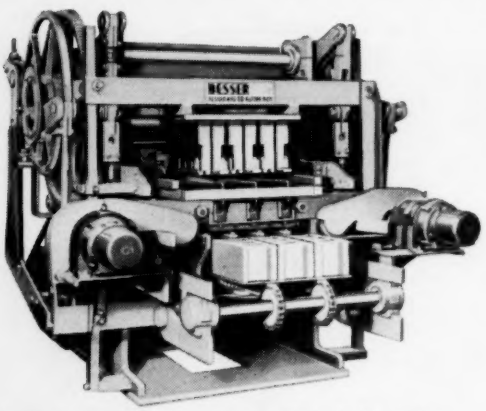
FLOORS - - - - - DRIVEWAYS - - - - - PAVEMENTS - - - - - ROADS

Booklet, "Concrete Facts about Concrete Floor Finishes" and full information sent on request.

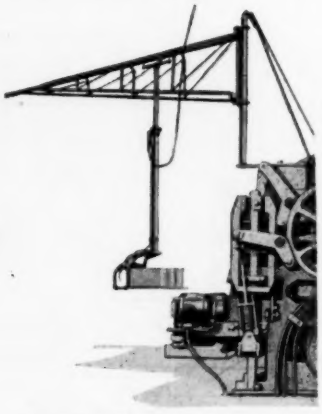
BUFFALO 287 Hinman Ave. NEW YORK

BESSER VIBRAPAC PLAIN PALLET STRIPPERS

Undirectional Vibration for compacting concrete blocks (Flam and Besser patents) combined with Besser patented principle of making all sizes on one set of plain pallets.



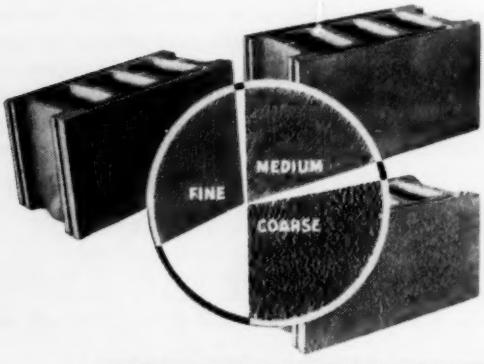
Besser Super 100% Automatic Plain Pallet Vibrapac. Capacity 600 8" x 8" x 16" blocks per hour, made 3 at a time on one plain pallet. Smaller units made in multiples of 4 to 32 (brick) on the same pallets.



Power offbearing hoist — One man offbears 600 8" blocks per hour. No lifting—Simply guide the hoist.

For Big Production Demand

The Besser Super Automatic Plain Pallet Vibrapac operates continuously making 600 blocks every hour throughout working period. Pallets are cleaned, sprayed with soluble oil and fed to machine, all automatically. No machine operator required.



Control of Texture and Density

The Texture and Density of blocks made on the Besser Vibrapac can be controlled to make them best suited to the particular purpose for which they are to be used. This flexibility is of great importance now that concrete blocks are used so extensively in every type or kind of building construction to save critical building materials.

WRITE FOR INFORMATION

BESSER MANUFACTURING COMPANY

COMPLETE EQUIPMENT FOR CONCRETE PRODUCTS PLANTS

800 43rd Street - - - - - - Alpena, Michigan

THIN SHELL CONCRETE ROOFS

LESS STEEL with BALANCED DESIGN

Total weight of reinforcing only about ⅓ to ½ of amount required for comparative structural steel design.

CONSTRUCTION SPEED and LOW COST

Smooth underside of roof permits formwork to roll on own wheels for many reuses and remarkable savings in cost.

GREATER SPANS for MORE USEFUL SPACE

Beam action of curved roof utilized to save materials.
Fewer columns and improved lighting and ventilation.

ADAPTABILITY and PERMANENCE

Defense plants, warehouses, hangars, etc., built with speed and economy. Less vulnerable to bombing, sabotage and fire.

ROBERTS AND SCHAEFER COMPANY
ENGINEERS
307 North Michigan Avenue, Chicago, Ill.
Eastern Branch Office: 1711 Connecticut Ave., N. W.,
Washington, D. C.

The vital part of any batching plant is an accurate, dependable batcher. Heltzel Weighing Batchers meet all specifications and are used for batching materials for all types of concrete construction . . . Highways . . . Airport Runways . . . Concrete Buildings . . . Block Plants . . . Dams . . . Sewers . . . Tunnels . . . Subways . . . Arsenals . . . Marine Bases . . . Docks, etc.

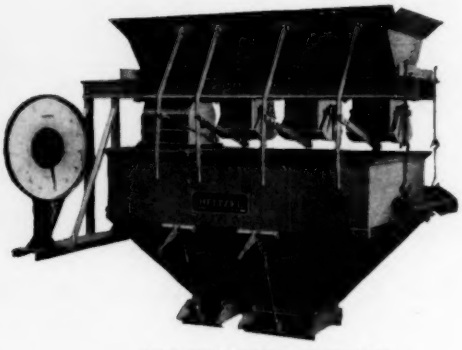
Heltzel Batchers are suspended on either dial or beam type scales made to Heltzel specifications by nationally known scale manufacturers and can be arranged for batching batch trucks, truck mixers, agitators, concrete buckets, stationary mixers, pumpcrete hoppers, etc.

The illustration at the left shows a typical Heltzel Batching Plant for charging Truck Mixers and the illustrations below the batchers available for use on Heltzel Batching Plants.

Write for additional information and descriptive literature.

UNIVERSAL BATCHER

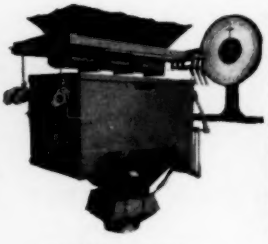
Interchangeable Hoppers for either Batch Truck or Mixer Charging.



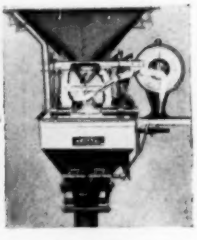
COMBINATION BATCHER

For Batching ALL Aggregate and Cement in One Operation.

2 or 3 Compartment Batcher Used where Specifications Call for Material to be Separated.



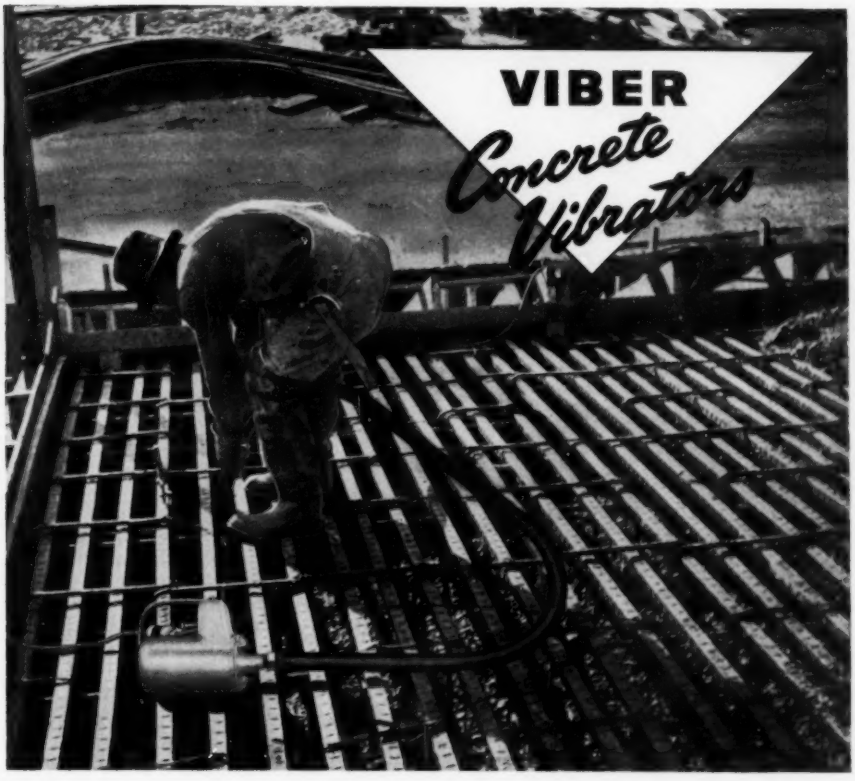
TRUCK MIXER BATCHERS
2 to 6 cu. yd. CAPACITY



HELTZEL DUSTLESS BULK CEMENT BATCHER

BULLETIN C-37

HELTZEL STEEL FORM & IRON CO.
WARREN, OHIO · U. S. A.



FOR ALL CONCRETE STRUCTURES

POWER

Viber built, electric motor, 110 volts A.C. or D.C. Rating 1¾ H.P. at 9500 R.P.M. Push and pull 30 ampere switch mounted in motor housing, protected from moisture. Net weight of motor 35 pounds. Pneumatic and Gasoline Motors also available.

FLEXIBLE SHAFTS

1¾ inch diameter built in 6 ft., 12 ft., and 21 ft. lengths. Viber built casings. Gamma spring wire track covered with multiple laminations of wire braid (280,000 lb. tensile strength) and spirally wound fabric. $\frac{3}{16}$ inch rubber covering. Stretching eliminated. No outside reinforcing to catch on forms.

VIBRATORS

1¾ inch, 2⅛ inch and 3 inch diameter. Approximately 11 inches long. Heat treated alloy steel housings. Rotating weight mounted on ball bearings running in oil bath. Vibrating speed in concrete 9500 R.P.M.

Complete interchangeability of Flexible Casings and Vibrators allows the use of the largest diameter vibrator permitted by steel spacing.

Distributors located in principal cities in the United States and many foreign countries.

VIBER COMPANY

726 South Flower St. **Burbank California**

Does PRECISION CONCRETE Control Pay?

The OWNER? The ENGINEER?
The CONTRACTOR? The ROAD BUILDER?
The CONCRETE MANUFACTURER?

The answer is YES for every one, based on experience. And this is why:

The $(SC)^2$ Moisture Meter. The only unit capable of determining moisture content in less than 1 minute

The $(SC)^2$ Compensator which compensates for the moisture content in every ingredient and graphically records its every operation

$(SC)^2$ Precision Control enables the concrete manufacturer, whether in a ready-mix or job plant, to make *better concrete* at a *lower cost* and all concerned share the benefits.

$(SC)^2$ CONTROL PROVIDES * * *

A MOISTURE DETERMINATION, accurate to ¼% by weight, made in one minute; *one for every batch when necessary.*

AUTOMATIC COMPENSATION, for all ingredients, for every change in moisture content of aggregates. Delivers not only the correct total quantity of water but also the correct DRY weight of each aggregate within close tolerances.

AN AUTOMATIC GRAPHIC RECORD, showing the delivered weight of all ingredients, and the moisture compensation made in each. An invaluable protection to both buyer and seller of concrete and a dependable aid to inspectors and engineers.

$(SC)^2$ CONTROL is used in progressive concrete manufacturing plants. Its precision mix assures rigid adherance to the engineers' specifications and produces remarkable uniformity.

Send for our booklet "PROFITS IN CONCRETE" and let us show you what $(SC)^2$ can do for your plant.

SCIENTIFIC CONCRETE SERVICE CORPORATION
MC LACHLEN BUILDING • WASHINGTON, D. C.

SCIENTIFIC $(SC)^2$ CONCRETE

$(SC)^2$ *is to concrete as sterling is to silver*

THIS BOOK...

Giving you helpful facts about concrete form work, plus how "Richmond" helps you do better, faster, more profitable jobs . . .

IS YOURS FOR THE ASKING

RICHMOND SCREW ANCHOR COMPANY, INC.

816 LIBERTY AVENUE • BROOKLYN, NEW YORK

TIME AND LABOR SAVING CONSTRUCTION DEVICES FOR CONCRETE CONSTRUCTION

This Annual Jaeger Report Concerns *Progress in Production*

During the past year, the entire energies of The Jaeger Machine Company have been employed in building equipment for the Army, Navy, Marine Corps and Maritime. We are proud to report that our progress, therefore, does not concern new methods or machines but new records of production, on vital airports and strategic roads, *which have been made possible* by methods originated and developed by Jaeger-Lakewood engineers.

TYPICAL EXAMPLE. The Jaeger Screw Spreader, shown above, is spreading and compacting 75,000 cubic yards of concrete of minus 1" slump, in 25 ft. lanes of 9"-6"-9" cross section for bomber runways, at an average day-after-day rate of 250 linear ft. per hour. The Jaeger Type "H" Finisher, shown below, is completing the operation behind the Spreader at this same rate of progress. This was an average of over 120 cu. yds. an hour. Under optimum conditions production reached 135 cu. yds. an hour.

It is a fact, recognized by the paving industry, that the amazing schedules of production established, in building America's wartime airfields and bases (of which the above is an example), could not possibly have been achieved without these two Jaeger-Lakewood developments — the mechanical finisher, first introduced by Lakewood in 1917, and the mechanical concrete spreader, first introduced by Jaeger in 1931.

PERFORMANCE OF JAEGER SCREW SPREADER:

The fundamental contribution of the mechanical spreader has been to furnish contractors with a practical means to distribute dry, vibratory type concrete mixtures between paving forms at the rate established by the 34E dual drum paver. Numerous contractors testify that the problem of spreading these harsh mixtures, in the 25 ft. width that is standard for most runway construction, would be insuperable without power spreaders. This is even more obvious in half width construction, where there is much less room for hand shovelers to work.

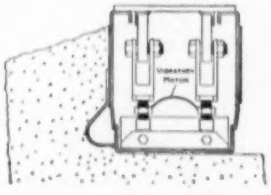
Further positive benefits, deriving from this Jaeger development, are (1) the elimination of segregation and the increased uniformity and density of slab texture resulting from the re-working of the material under the spreading screw and its firm compaction against the forms; (2) saving in man-power by removing shovelers from the pit and reducing final finishing time; (3) improvement of the final surface by relieving the finishing machine of need for bucking piles and striking off, thus permitting the finisher to concentrate on the riding surface.

PERFORMANCE OF JAEGER TYPE "H" FINISHER:

Wartime schedules for airport runways and military roads have placed demands on finishing machine capacity beyond anything hitherto encountered. For an example of the enormous volume of this production, Koss Construction Company of Des Moines, Ia., users of Jaeger-Lakewood Finishers, last year laid 2,000,000 sq. yds. of concrete pavement between January 1 and November 20. For an example of the tremendous capacity of the Type "H" Finisher on this type of work, contractors on an Ohio glider base finished 6300 ft. of 25 ft. wide slab in a single 21-hour day. We believe this to be a world record.

JAEGER VIBRATORY FINISHING is Also the First and the Unfailingly Successful Method:

In 1930 Jaeger-Lakewood introduced the first vibratory finisher ever built. The method (vibratory motors on bull-nose front screed) insures deep internal vibration of the mass — in contrast to mere surface vibration. It has unfailingly met every test by Highway Departments.



THE JAEGER MACHINE COMPANY, COLUMBUS, OHIO

(Lakewood Engineering Company, a Subsidiary) — Manufacturers of Concrete and Bituminous Spreading and Finishing Equipment, Form Tampers, Road Forms, Concrete and Bituminous Mixers, Truck Mixers, Pumps, Hoists, Marine Winches.

Effective
WATERPROOFING COMPOUNDS

☆

PLASTIMENT—*The Concrete Densifier*

SIKA Products are made entirely in this country by an American Company, using formulae worked out by the well-known Swiss firm of the same name. Many prominent government and private projects in the United States have been constructed with the help of SIKA waterproofing compounds or PLASTIMENT. Other structures which had developed serious leaks or infiltration of dampness have been remedied by SIKA maintenance waterproofers. SIKA Products for the concrete field come in three broad groups, briefly discussed below. Technical advice will be gladly given.

Sika
Integral Waterproofing Compounds

There are several of these compounds, each best for certain classes of work. In general, they are used on concrete and brick mortar to provide maximum impermeability from the start. Integral or initial waterproofing is always better than remedial waterproofing later on.

Sika
Waterproofing Compounds for Maintenance

These are in the quick-setting class and are used only for remedial work. By proper application of the correct compound, even heavy streams can be checked. It requires no special skill to apply these SIKA Products.

PLASTIMENT
The "Vitamin" for Concrete

This densifier has been used on several large government projects in the past year or two. PLASTIMENT improves the strength, durability and impermeability of concrete and eliminates faulty construction joints and honeycombing. When added to the concrete mix, it increases the plasticity and workability; it strengthens the bond with the reinforcing steel.

Other SIKA Products include: Expansion Joint Compounds, Floor Surface Hardeners, Tile Setting Compounds.

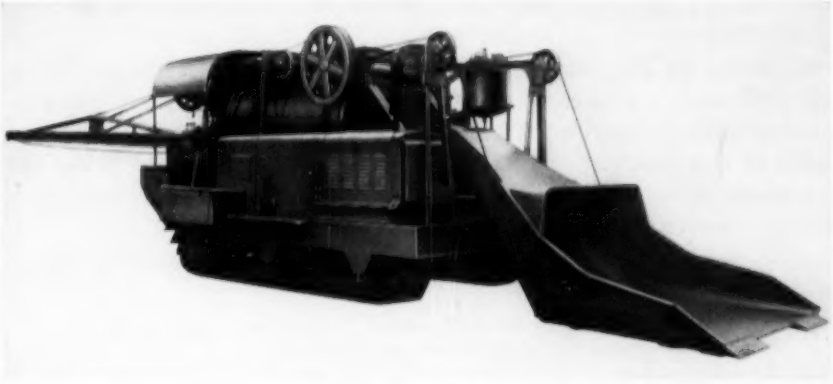
For details, consult Sweet's Architectural Catalog

SIKA, INC. 37 GREGORY AVENUE PASSAIC, N. J.

12—SI—1

Advantages of the Ransome Paver

Hydraulically Operated Boom Bucket Gates



In the field for about two years, the Hydraulically Operated Boom Bucket Gates of Ransome Pavers give operators these time and money saving advantages.

Eliminates split batches on narrow roads and at expansion joints.

Permits discharge of any part of bucket load at any bucket position on boom without returning bucket to Paver. Boom can then be swung to next slab to discharge the balance. Thus, each time a split batch is avoided a batch in mixing time is saved.

Clearance from ground to bottom of open bucket gates is 25 inches for clearing template pullers and scratch boards.

Bucket gates can be opened to any degree at any position on the boom.

A batch of concrete can be spread completely across a 25 foot runway with paver working outside of form.

A simple foot pedal operation controls opening of bucket gates to any desired degree. Gates are tightly closed by spring action, making the return trip to Paver unnecessary as the conventional trip-action is eliminated.

Bucket has no inside mechanism or obstructions which leaves full opening to permit free flow of dry concrete.

Dripping concrete eliminated by one-half inch over-lap of bucket gates.

Bucket gates are operated through an independent cable eliminating wear on boom bucket travel cable.

Write for RANSOME *Literature*

RANSOME MACHINERY COMPANY

DUNELLEN NEW JERSEY

This is a paid advertisement by the Dewey and Almy Chemical Company

Some Observations on Durability

By HENRY L. KENNEDY*
Member, American Concrete Institute

SYNOPSIS

The effects upon concrete of air entraining agents and a catalyst-dispersing-agent are discussed, based upon available data and performance records.

"Concrete for Permanence" is a basic theme of our industry, but it was rather rudely shaken by the failure of some concrete highways subjected to calcium chloride surface treatment. As the result of subsequent studies of this problem, the freezing and thawing test for durability has been given increasing acceptance and is today recognized as one yardstick of concrete permanence outside of actual field experience.

Durability, as measured by resistance to freezing and thawing, can be obtained by the following means:

1. Proper aggregate gradation and cement content with low water-ratio, resulting in a concrete of high density and strength.
2. Air entrainment.
3. The finer structure of the cement matrix through the use of the catalyst-dispersing-agent TDA.

Let us look at the effect of air entrainment and catalysis plus dispersion on any given cement, when compared to that same cement in the absence of either type of addition agent:

DURABILITY (as measured by freezing and thawing)	Air entrainment (foaming agents)	Excellent measured durability, especially in presence of chlorides.
		Lowered density.
		Decreased compressive strength[1]
		Decreased bond strength[1]
	Catalyst-dispersing-agent (TDA)	Improved durability in presence of water
		High density
		Greater compressive strength[2]
		Equal or greater bond strength[2]
		Improved paste structure

[1]At reduced w/c for same slump.
[2]At same w/c and increased slump.

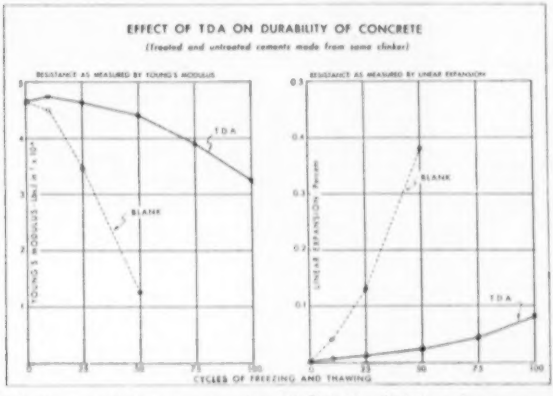
In the intensity of the search for improved durability for resistance to calcium chloride action, some of the other properties of the resulting concrete have not always been given adequate consideration. For general construction work, durability alone, as measured by calcium chloride resistance, is not enough. A weak structure lacking in compressive strength or bond strength may fail long before durability becomes an important factor.

Air entrainment in portland cement concrete, because of its resistance

*Manager, Cement Division, Dewey and Almy Chemical Company.

to scaling in the presence of calcium chloride, is well suited for highway construction. However, because of the loss in density, compressive and bond strengths, it is debatable whether this method of treating portland cement concrete should be applied to all type of general construction.

Where chlorides are not involved, the use of catalysis-dispersion gives improved durability without sacrifice in density, compressive or bond strength. And this improvement is noted with cements even where the use of TDA as a grinding aid is as low as 2 oz./bbl. The following curves show the effect of TDA when used in these small amounts. The improvement shown represents freezing and thawing in the presence of water.


EFFECT OF TDA ON DURABILITY OF CONCRETE
(Treated and untreated cements made from same clinker)
RESISTANCE AS MEASURED BY YOUNG'S MODULUS
RESISTANCE AS MEASURED BY LINEAR EXPANSION
TDA
BLANK
CYCLES OF FREEZING AND THAWING

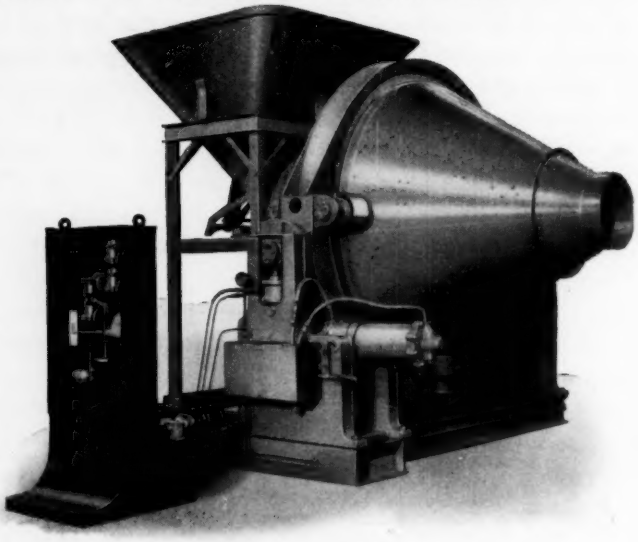
Where TDA is used in amounts of ¾ lb./bbl. of cement at a cost of 6-8¢/cu. yd. of concrete, unparalleled increases in workability will be obtained with no sacrifices in strength or bond or without entraining excessive air, the other qualities being still further improved. This amount of TDA will in general reduce the water-cement ratio 10-15%.

Summing up these observations, it appears that in any consideration of the practical aspects of durability, the mechanism for accomplishing it is most important. Until long-time data and performance records are available, durability alone, as measured by freezing and thawing, should not be considered the only criterion of concrete excellence. While air entrainment seems to have real merit in certain special cases, it does impose limiting properties. On the other hand, dispersion alone is not enough; catalysis is necessary to supplement and take advantage of dispersion. TDA, because it combines catalytic and dispersing action, enhances those desirable properties essential to "Concrete for Permanence."

What is TDA?—TDA is a mixture of triethanolamine salts and highly purified soluble calcium salts of modified lignin sulfonic acid. It has two functions: (1) catalytic action; (2) dispersing action. By means of its catalytic action it increases the reaction rate between portland cement and water, taking advantage of the greater surface areas exposed by the dispersing action. Factual information available to any interested concrete engineer.

Sometime in 1943, when the final results of our long-range testing program have been compiled, the Dewey and Almy Chemical Company intends to announce a new air-entraining agent.

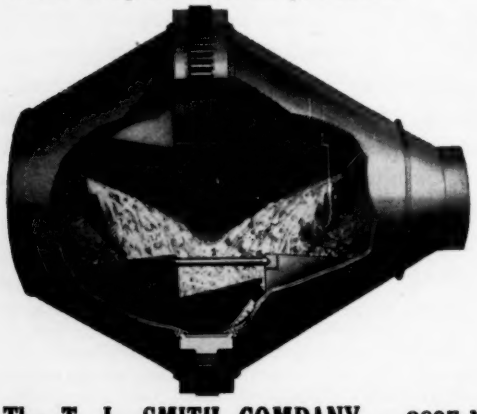
SMITH TILTING MIXERS



View of a 2-yard Smith Tilter equipped with gated batch hopper, pneumatic tilting device and consistency (workability) meter. Also available in 1-yard, 3-yard and 4-yard sizes.

THE reputation of Smith Tilting Mixers among Engineers and Concrete Technicians for thorough mixing, fast charging, and quick "Tilt and Pour" discharge without segregation, has been earned by their performance during the past forty-three years. Smith originated, developed and perfected front end charging in 1935, and now Smith Tilters can be equipped with Consistency or Workability Meters, thus enabling the Concrete Technician to determine at a glance the results which are being recorded for a permanent record.

"End to Center" Mixing Action—The Smith duo-cone shaped drum, plus the V-shaped mixing blades, produce specification concrete within the specified mixing period, and the results are not subject to variations in the charging sequence. On Smith-mixed jobs, large aggregates, or cobbles, can be used in the mix without detrimental increase in the fineness modulus of the designed mix. The distribution of the fine and coarse aggregates, sand cement, aggregate mortar and water cement ratios of samples from the front, center and back of the drum, will be well within the requirements of the specifications.



Consistency or Workability Meters—The Smith Consistency or Workability Meter was designed specifically for use on Smith Tilters. It is the only device available on any mixer which is not subject to errors in batching, residual concrete in the drum from batch to batch, frictional resistance of the mixer bearings, or variation in the weight of the different mixers in the plant. the performance of each of which is at times recorded on a single chart.

The meter operates through the medium of hydraulics. The tilting frame, which carries the drum, is supported on hydraulic cylinders located in the pedestals, one at each end of the tilting frame.

The T. L. SMITH COMPANY, · 2897 N. 32nd St., Milwaukee, Wis., U. S. A.

A3932-2P

SMITH TILTING MIXERS

The pressure created in these cylinders, by the tilting frame and the elements supported by it, is transferred to two sets of small hydraulic cylinders in the pantograph case. These pressures are added in one set of these cylinders and subtracted in the other set. To measure accurately the resistance of the concrete to the action of the V-shaped mixing blades, the forces due to the mixer itself are eliminated. This is accomplished by counter-balancing the set of cylinders which adds the pedestal reactions, to cancel the weight of the mixer—and by counter-balancing the set of cylinders which subtracts the pedestal reactions to compensate for the difference in weight of the two ends of the tilting frame, one end of which carries the power and drum drive transmission. The pedestal reactions, due to the concrete batch, are proportional. These reactions are measured to determine the horizontal movement of the batch due to the action of the mixing blades. This movement is transmitted to the recorder and the Visible Gauge by means of Selsyn Motors. The movement of the batch is indicated by a rotating indicator hand travelling over a chart of approximately 36″ circumference. On this chart the desired reading for each class of concrete required by the specifications has been calibrated. The zero position of the chart for each classification of concrete is brought into proper relation with the indicator hand by rotating the chart. This makes it possible to distribute the calibrations for the different mixes so each can be read and interpolated at a glance. The permanent record of each batch is, if desired, recorded on a travelling tape, the width and speed of which makes the record easily readable.

SMITH - MOBILE TRUCK MIXERS

Smith - Mobile's exclusive VISIBLE MIXING feature permits consistency control through observation of the entire batch during the mixing process. On the Smith-Mobile truck mixer the feed chute sealing door, which closes the drum opening, can be opened wide at any stage of the mixing cycle to observe the "end - to - end" mixing action. The Smith-Mobile drum revolves during the charging operation so the shrinking and mixing process starts instantly.

Smith-Mobile truck mixers were used on the construction of the Contra Costa Canal of the Central Valley Project under the supervision of the Bureau of Reclamation. The specifications limited the slump change to a maximum of ½″, and the water-cement and sand-cement ratios of samples from the front, center and back of the drum to a maximum variation of 10%. The performance of the No. 4 Smith-Mobiles under these rigid specifications is indicated by the following typical field records.

MIXER EFFICIENCY TESTS — SMITH-MOBILE MIXERS

Central Valley Project—Delta Division—Contra Costa Canal

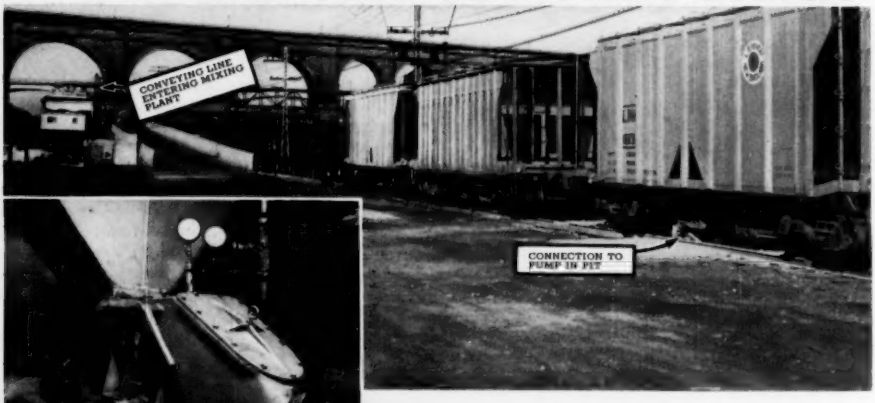
"Simultaneous Batching" ¾ max. canal lining concrete. 1:2.5:3.4 field mix.

Test No.	% of Lowest G/M			% of Lowest W/C			% of Lowest S/C			Slump			No. Revs.	Remarks
	Front	Center	Rear	Front	Center	Rear	Front	Center	Rear	Front	Center	Rear		
1C	104.0	100.0	100.0	100.0	103.0	100.6	101.5	101.5	100.0	1¾	1½	1¾	39	Good placeability
2C	108.8	100.0	103.5	101.1	100.0	100.3	100.0	103.6	102.7	2⅛	1¾	2¼	40	Good placeability
4C	102.0	100.0	131.9	104.9	105.3	100.0	100.0	105.7	101.3	2	1½	1½	49	Good placeability
5C	106.7	100.0	102.5	100.0	101.2	100.5	100.0	103.2	103.2	2⅛	1⅞	2	46	Good placeability
3C*	109.3	107.5	100.0	101.3	100.0	103.2	110.1	100.0	116.8	1¼	1⅜	1⅜	42	Good placeability

*"Dry Batching" ¾″ max. canal lining concrete. 1:2.5:3.4 field Mix.

The T. L. SMITH COMPANY, 2897 N. 32nd St., Milwaukee, Wis., U. S. A.

FULLER-KINYON . . . THE CLEAN, SAFE, LOW-COST CONVEYING SYSTEM



FOR CONVEYING DRY PULVERIZED MATERIALS THROUGH PIPE LINES

SOME MATERIALS CONVEYED BY FULLER-KINYON SYSTEMS

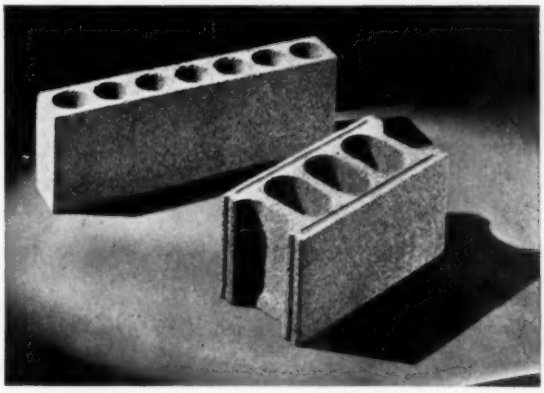
Arsenic dust
Asphalt filler dust
Bag fume
Baroid
Barytes
Cement (Portland)
Cement raw materials
Clays (dried)
Colox
Copper converter dust
Dextrine
Dolomite
Fullers earth
Gypsum (calcined)
Gypsum (raw)
Lime (quick)
Magnesite
Manganese Dioxide
Ore (pulverized)
Rock dust
Siliceous Powder
Starch (Pearl)
Starch (Powdered)

●

The above photographs show how a ready-mixed concrete plant takes advantage of the Fuller-Kinyon System. Cement is received in hopper-bottom cars; the car is "spotted" directly over a Fuller-Kinyon Pump installed in a pit underneath the tracks. From this point the pump transports the cement to the mixing plant, a distance of 700 feet. The smaller photograph shows the Fuller-Kinyon Stationary Pump installed at this plant.

Power, installation, maintenance and operating costs of an average system considered, Fuller-Kinyon is lower than those of any other. It can be operated with a minimum of supervision. Conveying through standard pipe lines, layouts are extremely simple. Lines can be hung overhead or buried underground. Materials can be pumped economically far beyond the practical limits of mechanical systems. Distribution can be made to any number of bins to any remote part of the plant. P-58

FULLER COMPANY
CATASAUQUA—PENNSYLVANIA
CHICAGO—Marquette Bldg. SAN FRANCISCO—Chancery Bldg.



STEARNS
has four duties

1. To make maximum contribution to the war effort.

   The plant is working 168 hours a week on parts for war equipment entirely foreign to concrete machinery.

2. To keep Stearns machines in the field operating at top efficiency.

   A live, active Service Department is helping Stearns customers get the best service from their present block and brick making equipment.

3. To furnish new concrete products plant equipment when needed for necessary construction work.

   Such machines are being furnished from stock to those United Nations requiring them for the war effort.

4. To design for the future the concrete products plant equipment the Industry will need when peace comes.

   Stearns Engineers are exploring exciting new possibilities in concrete products manufacture to meet the accumulated demand for masonry units that will be released after the war. The concrete block machine of the future—the one now in process of development—will probably exploit more fully the vibration method of consolidating concrete and will reveal unprecedented capacity for volume production.

Stearns Manufacturing Co., Inc., Adrian, Mich.

Pioneers in the development of equipment for the volume production of concrete masonry units by vibration. Manufacturers of vibration and tamp type block and brick machines, mixers, skip loaders.

QUALITY, SPEED AND ECONOMY

Your Money's Worth in Concrete

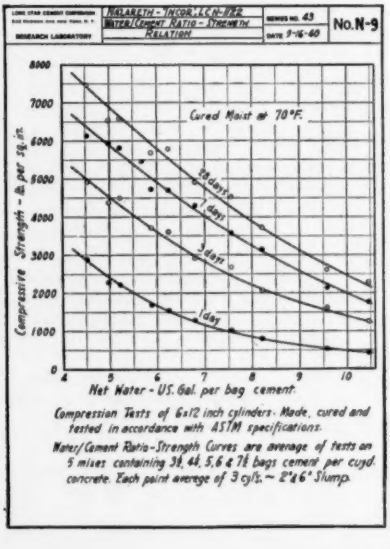
MODERN concreting demands modern cements, speed and economy in construction—and durability in service. That means selecting, for each part of the job, the type of cement which produces the lowest concreting cost—forms, time and cement all considered.

Between them, Lone Star Cement and 'Incor' 24-Hour Cement cover the entire range of building problems:

Lone Star for normal schedules, where time for curing and hardening is available;

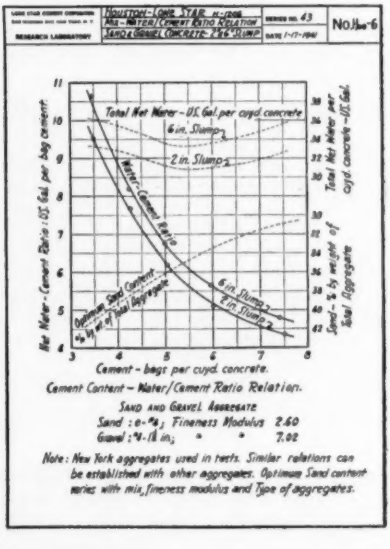
'Incor' where reduced curing time and 24-hour service strengths provide important construction economies. Both cements produce concrete of outstanding durability.

Let job analysis decide which cement to use. Selective concreting is the best way to get your money's worth in concrete.



Compression Tests of 6x12 inch cylinders. Made, cured and tested in accordance with ASTM specifications.

Water/Cement Ratio-Strength Curves are average of tests on 5 mixes containing 3½, 4½, 5.6 & 7½ bags cement per cuyd. concrete. Each point average of 3 cyls. ~ 2"&6" Slump.


Total Net Water - U.S. Gal per cuyd. concrete
6 in. Slump
2 in. Slump
Water-Cement Ratio
Optimum Sand Content
Net Water - Cement Ratio: U.S. Gal. per bag cement
Total Net Water per cuyd. concrete - U.S. Gal.
Sand - % by weight of Total Aggregate
Cement - bags per cuyd. concrete

Cement Content – Water/Cement Ratio Relation.

Sand and Gravel Aggregate
Sand: 0-¼ in. Fineness Modulus 2.60
Gravel: ¼-1½ in. " " 7.02

Note: New York aggregates used in tests. Similar relations can be established with other aggregates. Optimum Sand content varies with mix, fineness modulus and Type of aggregates.

Typical Performance Data

THROUGH SELECTIVE CONCRETING

Field experience since 1927 proves that 'Incor' can be used to advantage in all types of construction:

Structural Work: 24-hour form removal . . . high-speed schedules . . . 50 to 60% less forms. No reposting . . . ready for immediate use.

Heavy-duty Floors: Overnight service . . . superior wear resistance with only 24 hours curing. For severe exposures, cure 48 hours.

Watertight Concrete: Thorough curing in 1-2 days . . . saves 5-7 days' curing . . . helps assure watertight concrete. Smoother-working 'Incor'* concrete places easily, without separation.

Products Plants: Early strength means utmost utilization of equipment . . . minimum yard space . . . faster turnover.

Winter Construction: Now economical . . . only 1-2 days heat protection . . . savings on forms, heating, canvas.

To help contractors, engineers and architects design concrete mixes and plan construction schedules, Lone Star Research Laboratories furnish up-to-date performance data, as shown by typical graphs on opposite page. Continuous research and rigid manufacturing control assure you of the utmost in quality and uniformity.

*Reg. U.S. Pat. Off.

LONE STAR CEMENT CORPORATION

Offices: ALBANY · BIRMINGHAM · BOSTON · CHICAGO · DALLAS · HOUSTON · INDIANAPOLIS · JACKSON, MISS. KANSAS CITY · NEW ORLEANS · NEW YORK · NORFOLK · PHILADELPHIA · ST. LOUIS · WASHINGTON, D.C.

LONE STAR, WITH ITS SUBSIDIARIES, IS ONE OF THE WORLD'S LARGEST CEMENT PRODUCERS . . 15 MODERN MILLS . . 25-MILLION BARRELS ANNUAL CAPACITY

Carey Elastite EXPANSION JOINT

Authorities on the care of expansion joints agree that joints should be cleaned and poured with asphalt *when they are open the maximum amount.*

Joints should be maintained when necessary; but we do not consider it economical to spend the time and money in making sure that all joints are cleaned and poured when weather conditions reduce the volume of the concrete to a minimum.

We can't tell you how to avoid maintenance altogether; but we know that premoulded (extruding) asphalt joints are self cleaning and only need to be kept partially full of asphalt.

A well known engineer who has had to maintain thousands of miles of concrete pavement, in which premoulded asphalt expansion joints were used, says: "We are about convinced, however, that the filling of the openings adjacent to premoulded asphalt expansion joints is a waste of material. As long as there is a fair thickness of expansion joint material in the joint, we believe that material entering the crack along side of the joint will not cause spalling but will embed itself in the joint material and be extruded at a later date."

If you want to construct pavements and maintain them with a minimum of manpower and expense, use Carey Elastite (extruding) Asphalt Joint, the Joint which maintains itself

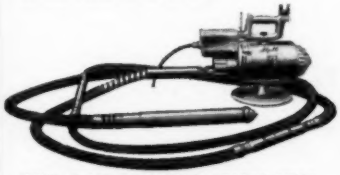
THE PHILIP CAREY MANUFACTURING COMPANY • Lockland, Cincinnati, Ohio

IN CANADA: THE PHILIP CAREY COMPANY, LTD. Office and Factory: LENNOXVILLE, P. Q.

MALL JACK-OF-ALL-TRADES POWER UNIT. An all-purpose tool for concrete vibrating and surfacing, sawing, sanding, wire brushing, drilling in wood, concrete, brick, iron or steel. Has 1½ H. P., single cylinder, four cycle, air cooled gasoline engine on low round steel base. Engine weighs 55 pounds and is easily carried by one man. Delivers a continuous flow of power for all jobs. You will find daily use for this big capacity, portable, production tool.

1½ H. P. Gasoline Powered Unit

MALL CONCRETE VIBRATOR POWERED WITH 3 H.P. GASOLINE ENGINE. Delivers 7000 vibration frequencies per minute. Has variable speed, single cylinder, four cycle, air cooled gasoline engine that runs all day on 1½ to 2 gallons of gasoline. The wheelbarrow type mounting provides unusual portability. Attachments can be furnished for CONCRETE SURFACING PUMPING, SAWING, SHARPENING TOOLS and BITS, FORM SANDING, DRILLING in concrete, brick, iron or steel.



MALL 3 H.P., 3 PHASE, GEARED HEAD CONCRETE VIBRATOR. Delivers 7000 vibration frequencies per minute. Can be furnished with various lengths of flexible shafting. This is an extra heavy duty unit adapted specifically for the vibrating of concrete in dams, locks, piers and other types of heavy construction.

MALL 1½ H.P. UNIVERSAL ELECTRIC CONCRETE VIBRATOR. Delivers 9000 vibration frequencies per minute under load. Like other MALL units, it is available with 14 ft., 21 ft., or 28 ft., of flexible shafting. Has low round base mounting and convenient carrying handle.

Hand puddling methods are no match for MALL Concrete Vibrators that place stiffer mixes faster with important savings in cement, sand, and labor. MALL vibrated concrete assures a better bond with reinforcement; free from honeycombs and voids; permits an earlier stripping of forms and makes stronger, water-tight jobs. MALL gasoline-powered units also operate quickly interchangeable tools for Concrete Surfacing, Pumping, Sanding, Wire Brushing, Sawing with Circular Saw, Drilling, Grinding and Sharpening Tools, making these very portable units easy to keep busy.

For their size, MALL concrete vibrators place more concrete than any other vibrator. The patented method of using a steel reinforced, high specific gravity, off balance lead weight revolving on ball and roller bearings inside of a totally enclosed steel protecting shell helps the vibrator to place concrete rapidly and effectively in the most inaccessible points of the forms.

ALL MALL vibrating elements are constructed of the toughest materials obtainable for maximum service and long life. Tips are welded with special metals to withstand the severe abrasive action which occurs during operation.

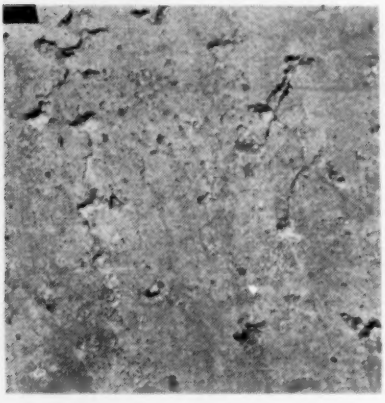
Write at once for full information!

MALL TOOL COMPANY
7703 South Chicago Ave. Chicago, Ill.
Offices and Distributors in Principal Cities

VACUUM CONCRETE Process

REMOVES from concrete, after placing, 15% to 25% of the excess mixing water—water required only for placing-plasticity but detrimental to quality.

ADVANTAGES—Formed surfaces free from air bubbles, see photos below.
Density and resistance to freezing and thawing improved.
Compressive strength 100% higher at 3 days, 50% at 28 days.
Improved resistance to wear.
Decreased volume change.



SHASTA DAM—U. S. BUREAU OF RECLAMATION

Typical concrete surface, **not** VACUUM-processed. Note unavoidable pitting due to water and air entrained in low-slump, 6" aggregate concrete.

VACUUM-processed surface. (VACUUM-mat construction built into contact face of forms) Removal of water and entrained air yields smooth surface, and harder, denser concrete, of higher compressive strength.

OTHER APPLICATIONS—Walls: The VACUUM-processed walls of a large Navy Dry Dock permitted removal of forms in 8 to 12 hours.

Slabs: Twenty-five million square feet of slab processed in 1942 on Army, Navy and Defense Plant Projects. Typical contractor's report shows 7,000 gals. water removed on average 40,000 sq. ft. daily pour; cement finishers off slab 2½ hours after last concrete placed; October-November temperatures.

No obligation involved in consultation to determine whether VACUUM CONCRETE can help with your concrete problems.

VACUUM CONCRETE, Inc.

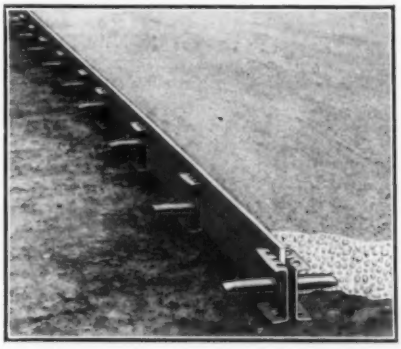
4210 Sansom Street - - PHILADELPHIA, PA.

'Phone—Baring 4545

HEL-CO *Modern Armor-Dowels*

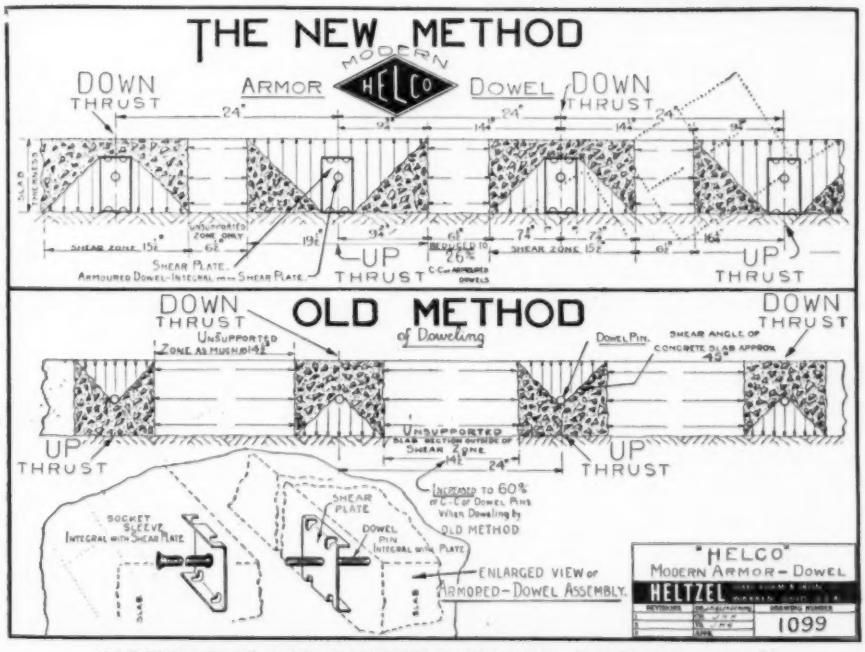
The NEW method of doweling for BOTH Longitudinal and Transverse Expansion & Contraction Joints in CONCRETE SLAB CONSTRUCTION . . . is already in use in State Highways and in Army airport paving and building floors.

SHEAR PLATE DESIGN GIVES 34% MORE SUPPORT THAN OLD METHOD
(See Drawing)



RIGHT: HEL-CO ARMOR-DOWELS in TYPICAL TRANSVERSE JOINT . . .

Write for Additional Information



LICENSED UNDER HELTZEL-WILLARD PATENTS . . .

HELTZEL STEEL FORM & IRON CO.
WARREN, OHIO · U. S. A.

Finishing Machines and Joint Installing Machines

Flexplane Ribbon Joint for concrete slab construction is automatically installed by the Flexplane Joint Installing Machine following the Finishing Machine. It is a permanent type of joint for concrete slab construction and less expensive to install, requires no maintenance and releases thousands of tons of vital steel for armament.

Ask for detailed information

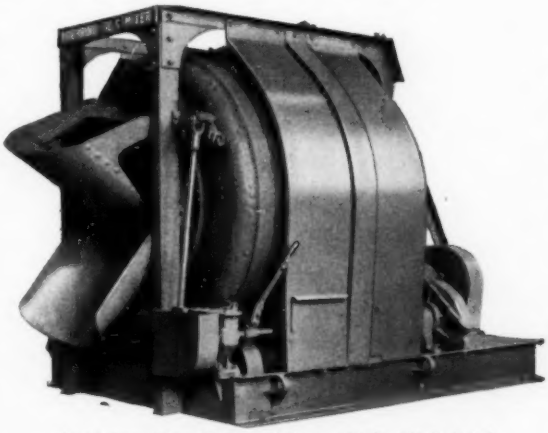
FLEXIBLE ROAD JOINT MACHINE CO.
WARREN, OHIO

KOEHRING

TILTING AND NON-TILTING HEAVY-DUTY CONSTRUCTION MIXERS

Koehring Concentric Zone Tilting Mixer 56-S, 84-S, 112-S

Important progress in the design of Koehring Tilting Construction Mixers during the past years has been made to further the production of better concrete . . . concrete best suited for the large volume dam structures and other similar projects. These improvements, because they are responsible for better concrete, also permit concrete production in a shorter mixing period. Large drum single opening, 15° above horizontal mixing position and 60° below horizontal discharge position are a few of the important progress changes in the design of Koehring Tilting Mixers.



Koehring Non-Tilting Mixer 28-S, 56-S, 84-S

Tilting mixers are built in sizes 2, 3 and 4 cubic yards of mixed concrete and can be used in concrete plants singly or in batteries of 2, 3, 4 or 5 using the concentric zone plan for discharge into one central hopper. Charging is also from one central group of bin and batchers.

Non-Tilting Mixers are available in sizes of 1, 2 or 3 cubic yards per batch. The Koehring Flow-Line discharge chute for minimum of segregation is one of many recent design improvements.

KOEHRING COMPANY - - Milwaukee, Wis.

POZZOLITH*

CEMENT-DISPERSING, WATER-REDUCING AGENT FOR CONCRETE

The great increase in interest on the part of engineers and builders in the Cement Dispersion principle has resulted from the use, since 1932, of Pozzolith on a rapidly increasing scale in both Private and Public construction. More than 45 major defense projects, as well as permanent public works of great magnitude and importance, have placed several million cubic yards of Pozzolith Concrete since 1939. Following is current information concerning Cement Dispersion and Pozzolith, the product in which cement dispersion is made available to the general construction market.

Pozzolith, unlike the materials commonly classed as admixtures, is not in its essential action an addition to the mix, but is a means whereby the cement itself may be used more effectively and more efficiently. Experience has proved that only by the introduction of the Dispersing Agent at the time of mixing the concrete are the fullest improvements in quality and economy secured.

Results reported from typical projects, and the findings of engineers and laboratories who have investigated the subject are included here.

FUNCTION OF POZZOLITH

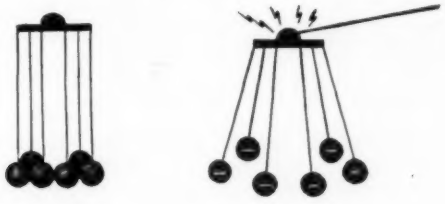
Microscopic studies of various types of portland cement in leading university and industrial laboratories have established the fact that when cement is placed in water the individual particles tend to clump together or flocculate. This flocculation reduces cement efficiency. Investigation shows that with 28-days curing, only 50% of the cement hydrates. [Anderegg and Hubbell, A. S. T. M., 29 II 554 1929]. Since portland cement depends upon *surface* reactions for its valuable properties, it is obvious that the efficiency of cement can be greatly increased by breaking up these agglomerations. This is the function of Pozzolith and is accomplished through the action known as Cement Dispersion.

---

*For composition of Pozzolith, see pages 171 and 172, J. A. C. I., Nov., 1940; Pozzolith identified as HR-O. Pozzolith is essentially a combination in powder form of the cement dispersing agent calcium-lignin-sulfonate, the electrolyte—calcium chloride, and the pozzuolana—low carbon fly ash.

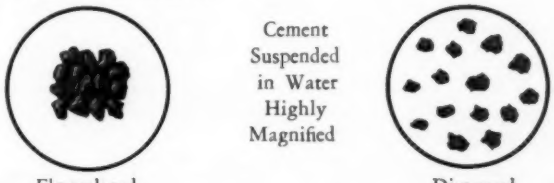
As a means of defining dispersion as it applies to cement in water, the following diagram is given:

FIGURE 1



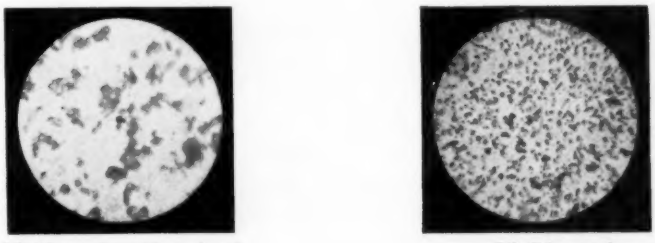
"Flocculated" Dispersed

Suspended pith-balls fly apart when given the same electrostatic charge. This is dispersion in air. This same electric force breaks up the flocculation of cement particles (as illustrated below) when the principle of dispersion is applied to cement in water.



Flocculated Dispersed

The electric force that makes the pith-balls mutually repellent likewise breaks up the flocculations of cement particles.



(a) Normal or Flocculated (b) Dispersed

Photomicrographs (a) of cement in water in its normal or flocculated condition and (b) of cement in the dispersed condition, produced by adding a small amount of dispersing agent to the water.

This Dispersion is accomplished in concrete mixes through the action of the calcium-lignin-sulfonate, component of Pozzolith which endows cement particles with the same electric charge. This is fully confirmed by reports of leading university authorities conducting studies in this field. By exposing a greater surface area of cement to hydration, dispersion improves the properties of concrete in both the plastic and hardened stages.

EFFECTS OF CEMENT DISPERSION IN CONCRETE

In Plastic Stage

Improved placeability with up to 20% less water.
Reduced segregation and bleeding.
Increased fattiness.
Greater water retentivity.

In Hardened Stage

Increased durability.
Increased watertightness.
Higher strength.
Greater uniformity and freedom from gross defects.

The effect of Pozzolith on the Durability of concrete as reported by L. R. Forbrich.

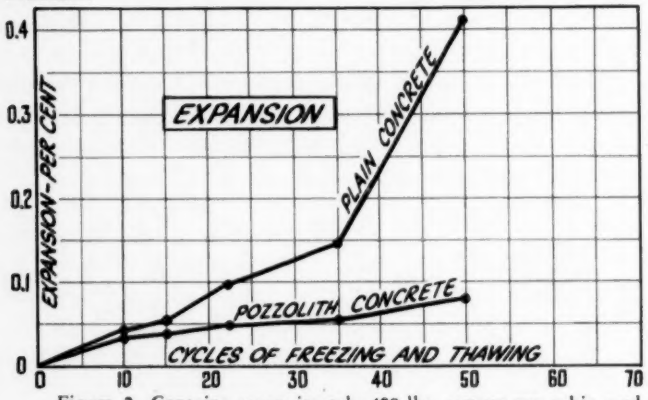
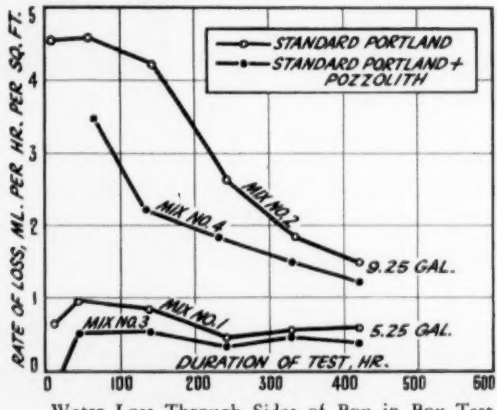


Figure 2. Contains approximately 480 lbs. cement per cubic yard. W/C. Plain concrete — 6.4 gal. per sack — Slump 2" W/C. Pozzolith concrete — 5.7 gal. per sack — Slump 2".

The effect of Pozzolith on the Permeability of concrete as reported by W. M. Dunagan, Associate Professor of Theoretical and Applied Mechanics, Iowa State College, Ames, Iowa. See "Methods for Measuring the Passage of Water Through Concrete", Pages 866-880, Proceedings of 42nd Annual Meeting, Volume 39, A. S. T. M. 1939.



Water Loss Through Sides of Box in Box Test.

ECONOMIC ASPECTS OF CEMENT DISPERSION

The results of field operations in several million yards of concrete since 1932 have established the economic value of Cement Dispersion. Users have found, for example, that the use of 5 lbs. of Pozzolith in a 5-sack mix produces results equal to or better than those produced by the use of an additional sack of cement.

In addition to reduced initial material costs, lower placing and finishing costs and the advantages of earlier form removal are obtained. Typical field reports showing this are given as follows:

Project Title	Type of Mix	Cement Factor Sacks—Per Cu. Yd.	Slump Inches	Compressive Strength Pounds per Square Inch 28-Days	Report by
Beck & Gregg Hardware Co.	Untreated Concrete	6¼	3½	4100	Georgia School of Technology for account of Atlanta Aggregate Co.
	Pozzolith Concrete	5	4	4275	
Marion, Ind. Sewage Treatment Plant	Untreated Concrete	6	6	4250	Pittsburgh Testing Lab. for account of Steigerwald and Borchert.
	Pozzolith Concrete	5	5½	4570	
Sewage Disposal Plant, Omaha, Nebr.	Untreated Concrete	7.0	4½	4422	Omaha Testing Lab. for account of Yant Construction Company.
	Pozzolith Concrete	6.0	3¾	4864	
Pennsylvania Power & Light Co., Kulpmont Station, Mt. Carmel, Pa.	Untreated Concrete	5.35	1¾	4638	Fritz Eng. Lab. for account of R. G. Crimm.
	Pozzolith Concrete	4.58	2¼	5390	

In the construction of a vehicular overpass between two plants of a large California Aircraft Company, the architects and engineers, Edward Cray Taylor and Ellis Wing Taylor report the following experience with Pozzolith Concrete:

Admixture	Gal. Water Per Sack	Slump	Sacks Per Cu. Yd.	Compressive Strength per Square Inch 7 Days	28 Days
None.....	6.25	2¾"	6.78	2400	3440
None.....	6.25	2¾"	6.78	2540	3815
Pozzolith...	5.79	4½"	6.05	2705	4405
Pozzolith...	5.87	3¼"	6.04	2545	4570

"Concrete was poured faster because of better workability of mix, thereby speeding up the work. Forms were stripped earlier than with ordinary concrete as strength of concrete was increased. Finishing of deck slab was done at a faster than normal rate."

Increased strength and slump with Pozzolith are reported as follows on the construction of 150 ammunition storage "igloos" in a Texas Ordnance Plant. Architect-Engineers: Gieb, LaRoche, Dahl and Chappell; General Contractors, Brown & Root, Inc.:

	PLAIN* Concrete 2½" slump	POZZOLITH* Concrete 4½" slump
7-day strength.....	1693 lbs.	2583 lbs.
28-day strength.....	3005 lbs.	4114 lbs.

*Five bags of cement per cubic yard were used in both plain and Pozzolith concrete. Strengths shown are average of 260 plain job-test specimens and 300 Pozzolith job-test specimens.

Important effects in hardened concrete resulting from the use of leaner Pozzolith mixes of equal water-cement ratio and consistency, include reduced shrinkage and less tendency to cracking due to lower cement paste content; reduced water content per unit of concrete which leaves fewer continuous channels, resulting in lower permeability; increased durability as measured by freezing and thawing.

METHOD OF USE

Pozzolith is manufactured in powder form under continuous and exact laboratory control from raw materials carefully selected to conform to our specifications.

Pozzolith is added to and mixed with the aggregates, water and cement, and the volume of water employed is reduced from that of an untreated mix to give the desired consistency. The routine established for handling and adding Pozzolith at the batching plant or mixer is a fully and easily controlled low-cost operation. Trained field men service new projects and new users.

For mortars, Cement Dispersion is supplied in a modified formula marketed as Omicron Mortarproofing.

LIST OF TECHNICAL PAPERS SENT ON REQUEST

"Application of the Principle of Dispersion to Portland Cement"—Research Paper No. 35.
"Economics of Cement Dispersion"—Research Paper No. 36.
"Relation of Dispersion to Special Cements"—Research Paper No. 37.
"Cement Dispersion and Admixtures"—Research Paper No. 38.
"Cement Dispersion and Air Entrainment in Concrete Pavement Construction"—Research Paper No. 39.
"Cement Dispersion and Concrete Floors"—Research Paper No. 40.

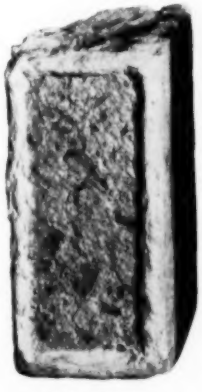
HP-7*

FOR CONCRETE RUNWAYS AND PAVEMENTS

Increased interest in the effect of air-entrainment on the durability of concrete, especially scaling resistance, has followed extended investigation in that direction. Experimental pavements laid in northern states with cement or concrete treated to impart high air content have developed increased durability and scaling resistance, but with a sacrifice in most instances of compressive and flexural strengths.

After extended study of the problem with the objective of preserving the essential qualities of the concrete while increasing air content, The Master Builders Company has developed HP-7, which markedly improves resistance to scaling and concrete durability without sacrifice in strength. In fact, with the same cement factor, compressive and flexural strengths are increased with HP-7. This is due primarily to the use of Cement Dispersion in combination with the air-entraining agent, Orvus. The use of HP-7 in a 5 sack mix produces strengths equal to those obtained in a plain mix using 10% additional cement. Furthermore, the other desirable properties produced through Cement Dispersion are secured.

FIGURE 3



Plain Concrete
5½ sacks per c. y., 3" slump

HP-7 Concrete
5.0 sacks per c. y., 3" slump

*HP-7 is essentially a combination of the air-incorporating agent, sodium lauryl sulphate, with the cement-dispersing agent, calcium lignin-sulfonate. The latter, marketed as Pozzolith, has been widely employed in general building construction since 1932, and is discussed in the preceding section. On sodium lauryl sulphate (Orvus) there is extensive laboratory and field performance data and its advantages are a matter of published record.

—continued on following page

The effect on durability and bleeding as illustrated by the results of freezing and thawing (with calcium chloride) tests is shown in Figure 3. The effects on flexural strengths with reduced cement factor, are shown in Table 3. Both Figure 3 and Table 3 pertain to concrete which was field cast in highway pavement near the Canadian border.

TABLE 3 — Flexural Tests on Paving Concrete With and Without HP-7

Type of Mix	Average Cement Factor Sks/Cu. Yd.	Average W/C gallons per sack	Average Unit Weight p. c. f.	Average Modulus of Rupture of 6 x 8 x 36 inch beams (1) 7 days	28 days
Plain	5.5	5.76	145	630	685
HP-7	5.0	5.78	141	630	685

(1) Each value for the plain concrete the average of 8 tests on 4 beams.
Each value for the HP-7 concrete the average of 16 tests on 8 beams.

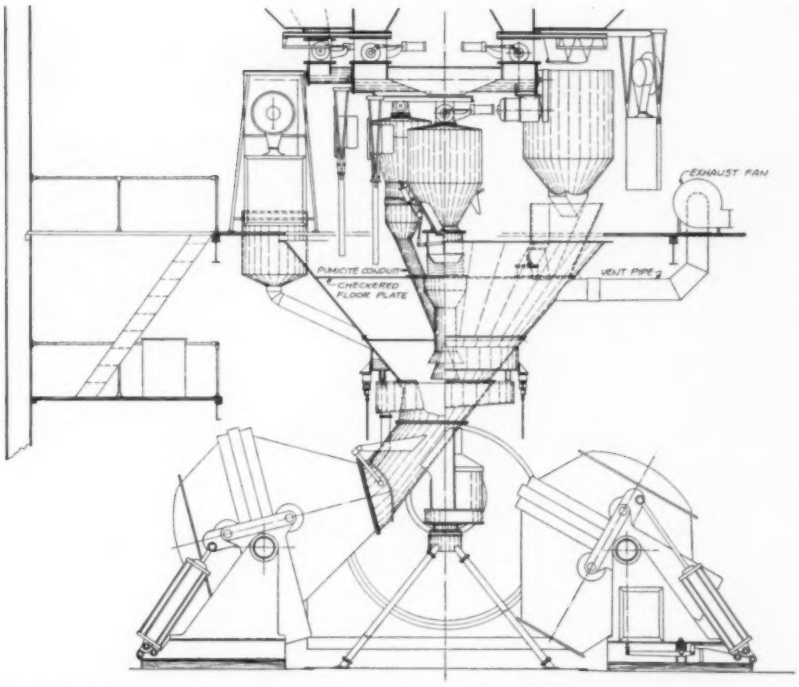
Field experience has shown that different cements ground with the same amount of air-entraining agents produced concrete with variable and unpredictable amounts of air. One published comment on this point, following extensive laboratory and field experience with both interground and added agents may be cited: "It might be pointed out that in construction operations those admixtures which could be added at the mixer were more desirable.—The reaction of the admixture to the mixes with aggregates from different sources will vary. Therefore, those admixtures which can be controlled in quantity at the mixer should prove to be more advantageous, since no corrective measure can be taken after an admixture is ground with the cement." ("General Observations on Concrete Scaling"—by J. W. Kushing, ROADS & STREETS, December, 1941)

HP-7 is supplied in the form of a powder which is added to the concrete when mixing in amounts predetermined to give the results required on the project.

May we take this opportunity to invite members of the Institute to visit our Research Laboratories and to call on us for additional data on the above subjects or any other of Master Builders several products and services?

THE MASTER BUILDERS COMPANY

CLEVELAND, OHIO **TORONTO, ONTARIO**



Johnson automatic central mix plants have been used by contractors in building the world's largest concrete structures. Boulder, Grand Coulee, Shasta, TVA projects and many others represent over 60 million yards of concrete proportioned by Johnson plants.

Some of the features which have made Johnson Central Mix Plants the choice of contractors who know: Concentric arrangement of bin with cement in the center for minimum of ground space; Full automatic control for accurate and uniform weighing of each batch; Johnson automatic system of batching for fast filling; Discharge control of each batcher regulated for ribbon feeding of material results in maximum pre-mixing of materials.

A Typical Johnson Automatic Plant

Bin size, 1350 cu. yds.
No. of aggregate compartments, 8.
No. of Cement compartments, 2.
No. of Mixers, 4—4 yd. tilter.

Batchers,	Material	Capacity
	Fine Sand	4000
	Coarse Sand	5000
	¼ to ¾	4000
	¾ to 1½	4000
	1½ to 3	5000
	3 to 6	4000
	Cement	3000
	Cement	3000
	Water	1500

Aggregate supply, 36" belt.
Cement supply, Bucket Elevator.

Typical mixing cycle of 4 cu. yd. mixers:

Charging time	10 sec.
Mixing time	120 sec.
Discharge time	8 sec.
Total	138 sec.

With 4 mixers, cycle per mixer—34½ sec.

Typical Batching Cycle:

Batcher filling	12 sec.
Batcher discharge	10 sec.
Total	22 sec.

Waiting time, 34½ — 22 = 12½ sec.

Swivel chute for charging mixers rotates from one mixer to another during filling time.

THE C. S. JOHNSON CO., - CHAMPAIGN, ILL.

PRODUCE BETTER CONCRETE WITH

Stripping Hydron from concrete on government dam.

FINER LOOKING . . . LONGER LASTING

HERE is a definite step forward in the improvement of concrete. U. S. Hydron removes water and air bubbles from the surface, solving two problems brought to the fore by the trend toward the use of mixes with low water-to-cement ratios.

This new absorptive lining not only eliminates surface imperfections but actually increases resistance to surface abrasion and to freeze-thaw action. And Hydron is inexpensive, easy to ship, to store and to apply.

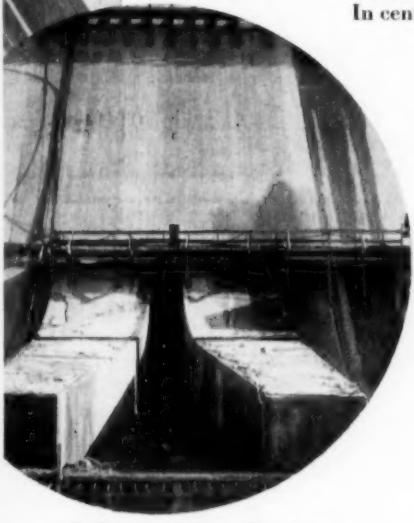
Tests indicate that the case-hardening effect produced by Hydron lining is at least 3 inches deep, and that there is a gradual change in water-to-cement ratio from the surface into the bulk concrete.

Hydron absorptive lining uses no critical materials.

UNITED

NEW FORM LINING HYDRON

On sides—concrete cast against plywood forms.
In center—concrete cast against Hydron.



Spillway of dam, showing use of Hydron on curves.

EASY TO USE

Thin and light—Hydron is only 0.08 in. thick; weighs 0.32 lb. per sq. ft.

Inexpensive to ship and store; easy to handle

Can be cut and trimmed with knife or scissors

Mounted to forms by use of rapid fire staple guns

No special forms needed; dimensional adjustments unnecessary

Flexible—fits curved or arched surfaces

Strips cleanly; no tedious finishing

Can be left on surfaces two weeks

If accidentally wet before use, can be easily and quickly redried

Send for your copy of informative booklet on Hydron

STATES RUBBER COMPANY

Mechanical Goods Division • Rockefeller Center • New York

VISUALIZING
Water Retention
. . . . in the Curing of Concrete

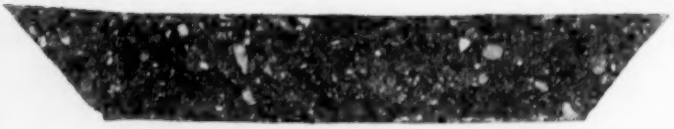


Fig. 1—Concrete Cured with Truscon TRU-CURE

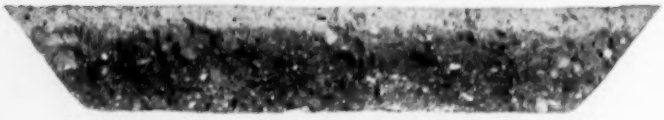


Fig. 2—Cured with Low Water Retention Curing Compound

WHAT THE "WATER INDICATOR" DISCLOSES about CONCRETE CURING COMPOUNDS

Photographs above were taken of slabs cast in "pie plates" from the same concrete mix—each treated with curing compounds as labeled, immediately after brooming. Kept in cabinet at uniform temperature of 100° F. and 30% relative humidity for seven days. Slabs were then broken, and fractured cross-sections sprayed with a "water indicator" to see how the moisture was distributed in the concrete. Tru-Cure's very high percentage of water retention is plainly shown by the uniform darkening of the concrete in Fig. 1.

Fig. 2 shows what happens when any of the earlier types of concrete curing membranes having low water retention are used on concrete. The "water indicator" shows moisture in the base, but near the surface water is lacking. (That section of concrete where the "indicator" did not find water appears light in color in the photograph) Near the surface is exactly where the concrete should have had ample water for proper cement hydration. Lack of water at this point means surface shrinkage, hair checks, cracks—a softer wearing surface—a poor concrete for roads, floors, and areas subjected to wear or weather.

TRUSCON IS A NAME long and favorably known in the concrete industry. For over thirty years, Truscon Laboratories have been formulating and manufacturing materials for concrete and concrete products.

TRUSCON TRU-CURE—

the modern, liquid concrete curing membrane, is one of a long line of successful treatments for concrete, engineered and produced by the Truscon Laboratories. Following is a list of Tru-Cure's more interesting properties:

1 **HIGH WATER RETENTION.** Laboratory tests on Tru-Cure run better than 96% water retention in first 24 hours, and over 90% in seven days (at 100° F. and 30 relative humidity).

2 **HIGH EARLY WATER RETENTION.** The fact that Tru-Cure is applied immediately after finishing (no waiting for cement to set) gives it at least an hour to five hours' start on curing methods such as burlap or cotton mats. This prevents moisture loss from the surface of the concrete during this very important period in the concrete's life.

3 **NO NEED FOR WET BURLAP OR WET COTTON MATS** as a preliminary treatment when Tru-Cure is used. Because of its dual properties of (1) very **high** and (2) **early** water retention, Tru-Cure retains all the mixing water necessary for the cement to thoroughly exercise its full normal function of combining with water.

4 **ALL THE EARLIER TYPES OF CLEAR CURING MEMBRANES** had such very low water retention that engineers were forced to insist that wet burlap or wet cotton mats be used for the initial 24, or even 72, hours of curing with such compounds. There is no need for this "preliminary curing" with Tru-Cure (see "water indicator" test, above).

5 **VERY LIGHT COLOR.** Tru-Cure is a very light colored liquid membrane widely used on roads, airplane runways, etc. A fugitive dye is usually added to assist in application.

6 **ONE OF THE FEW CURING COMPOUNDS ACTUALLY APPROVED BY U. S. WAR DEPARTMENT ENGINEERS.** Also approved by many State Highway Departments, County Boards, and other governmental agencies; meets the requirements of the Bureau of Reclamation and U. S. Public Roads Administration.

7 **CONTROLLED EVAPORATION OF WATER.** The results of numerous tests all accord Tru-Cure a very low and **uniform** rate of water loss. Contrast this with a fluctuating rate such as is occasioned by alternate saturation with water, and later drying, when burlap or cotton blankets are used. Concrete cast under laboratory conditions can be uniformly cured with wet burlap. But out in the field many things happen. The wind blows the blankets off frequently. Water is sometimes hard to get, the blankets dry, sucking water out of the concrete through capillary action. This results in a fluctuating water content, especially near the surface of the concrete. The effect of such alternate wetting and drying producing stresses and strains in the wearing surface of the concrete is pretty well understood by engineers.

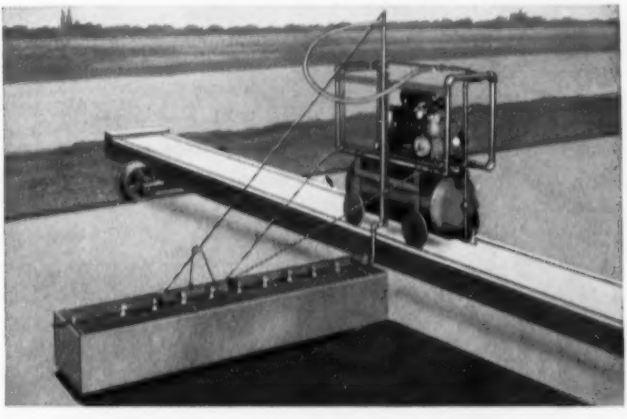
8 **TRU-CURE SAVES LABOR.** Once applied, no further supervision is required. Tru-Cure saves labor in application which today can be used to better advantage elsewhere. No cleaning up is required after the Tru-Cure curing process is over. Saves considerable in equipment such as stocks of burlap or cotton mats, the cost of laundering, drying, and warehousing same—the use of trucks for hauling, and pipe lines, etc.

9 **UNIFORM APPLICATION.** It is evident that irrespective of how efficient a curing membrane may be, unless it is applied to a film of proper thickness—and **uniformly,** so that every square inch of concrete is covered with a sufficient amount of water-tight film, it cannot effectively seal in moisture, thus uniform and adequate curing is not possible.

There has long been a demand for a convenient and reliable mechanical means for assisting the applier to maintain a proper application rate for his curing compound. To meet this urgent need Truscon's Engineering Department has designed an "Application Machine" as illustrated. With this equipment which may be procured at reasonable cost from equipment dealers, the contractor is able to provide himself, the project engineers, and the owners with the added assurance that the curing of the concrete will be uniform, adequate and effective.

It is true that thousands of square yards of concrete pourings have in the past been perfectly cured with Truscon Tru-Cure using ordinary spray equipment—nevertheless, a mechanical applicator such as this does eliminate much of the uncertain "human element." Contractors have asked for it, engineers have insisted that "something should be done about the application process." Truscon has provided it for the approval of the industry. Further information on the "Application Machine" may be obtained by addressing Truscon Laboratories.

Request for Literature on TRU-CURE, or inquiries on factual data pertaining to concrete curing, are invited.



Concrete Curing Application Machine Developed by Truscon Laboratories (Patent Applied For)

TRUSCON LABORATORIES, Inc., Detroit, Mich.

The Third Brother Goes

HE BELONGS to a large family of wonder workers.

They are men of faith, who perceive not only the difficulties, but the ways to surmount them.

The Oldest Brother of the family is more active today than ever before. He is over 5,000 years old, but every army drafts him. His name is Military Engineering.

The Second Brother also counts his years by the thousands, although his modern name is not more than 200 years old. His work is with great structures . . . with waterways, railways and highways . . . with lighthouses, harbors and airports. His name is Civil Engineering.

The Third Brother is a youth with only a century or so to his credit. He designs . . . constructs . . . applies, sells and maintains apparatus for generating, transmitting and using power. His name is Mechanical Engineering.

He founded this company 50 years ago. Here, under the name of Rex Mechanical Engineering—Rex M. E. —he has been working for the Second Brother—Civil Engineering—by providing better machines for mixing concrete, placing concrete and for removing water.

For the duration, he is working for both his Older Brothers—to help the Oldest finish his job and retire.

Now, as in the years of peace, he designs . . . manufactures . . . applies . . . sells and maintains machinery for concrete mixing and placing, and for water removal.

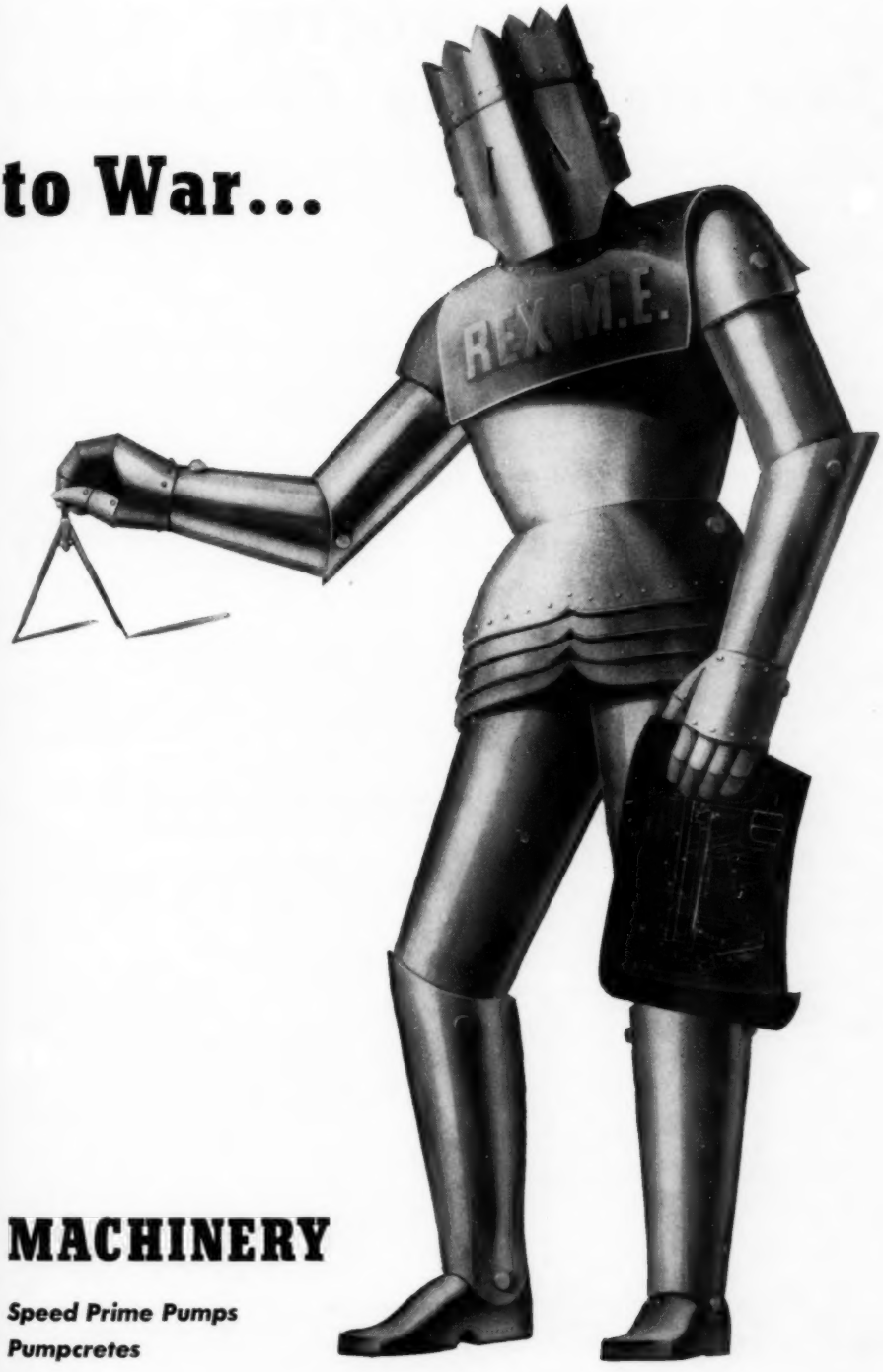
While working to bring V-Day nearer, Rex M. E. is learning and applying many things that are helpful now, and may be even more so afterward.

CHAIN BELT COMPANY OF MILWAUKEE

REX **CONSTRUCTION**

Concrete Mixers • Moto-Mixers
Pavers • Mortar and Plaster Mixers

to War...



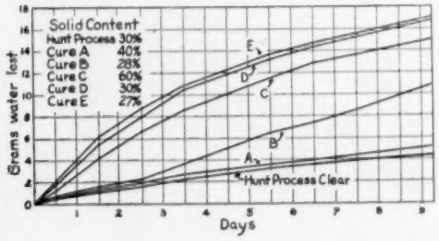
MACHINERY

Speed Prime Pumps
Pumpcretes

HUNT PROCESS
Concrete Curing Compounds

Concrete will cure properly provided there is sufficient water for complete hydration of the cement evenly dispersed throughout its mass during the curing period. It is now generally recognized that this can be satisfactorily accomplished by application of an effective sealing compound, such as Hunt Process Black or Hunt Process Clear, to the surface of the concrete immediately after completion of the finishing operations or removal of the forms. There are wide differences in the sealing efficiencies of liquid curing compounds. These differences should be determined before use in order to establish the comparative rates of coverage necessary to afford satisfactory sealing action and to ascertain the actual curing cost per unit of surface. Such a determination can be made in the laboratory or in the field with inexpensive equipment.

Through our experience of many years in determining comparative sealing qualities of membrane cures we have found the following procedure to be simple and easily performed. It may be necessary for the novice to first test several series of specimens in order to acquire a technique that will produce consistent results. The relationship of the surface area of the mortar specimen to its water content is such that the least imperfection in the coating due to improper application will result in high water loss not correctly attributable to the sealing compound itself.



Procedure for Determining Comparative Sealing Qualities of Liquid Concrete Curing Compounds

Portland cement mortar having the same proportions of water, cement, and sand as the concrete to be used shall be thoroughly puddled into containers having sides at right angles to the bottoms, or nearly so.

> *Note:* The proportions specified have been adopted by a number of public bodies as standard procedure. However, the resulting mortar usually contains a somewhat excessive amount of water. We recommend proportions of 12% water, 26% cement, and 62% sand. The sand should be dry and uniformly graded and should all pass a No. 8 screen. For type of container we recommend an 8-ounce, seamless, gill-type, flat pattern, tin ointment can, which, however, can be used only once. Cake tins with sharply sloping sides have the advantage of being reusable.

After filling the containers the mortar shall be struck off slightly below the level of the top and shall be smoothed with a small steel trowel to a level surface which will permit the sealing compound to be applied uniformly and prevent it from flowing to low spots. The specimens shall then be placed where they will be subject to the free circulation of air and allowed to remain until the mortar has stiffened sufficiently for troweling.

> *Note:* It is desirable to have the mortar slightly below the rim of the container in order that a more positive seal can be obtained at the line of contact. The delay prior to troweling should be about two or three hours, which conforms to the interval between placing and finishing on the job. The fresh mortar specimens should be picked up carefully, exerting as little pressure as possible on the sides of the containers in order to prevent distortion and formation of a crack at the line of contact of the mortar with the container. Because of expansion of the container and the chance of cracks forming between the container and the mortar, due to rapid change in temperature or to handling, special care must be exercised to prevent a condition which will permit excessive evaporation of the mixing water.

As soon as all indications of excess moisture on the surface have disappeared, the surface shall be *lightly* brushed and

the curing compound applied at the rate of 200 square feet per gallon.

Note: The brushing of the surface is desirable to remove any laitance and to roughen the texture *slightly*. This roughening will assist in the even dispersion of the compound. The curing compounds should be applied at the same temperature (at least 70° F.). So far as practicable the specimens should also be approximately at this temperature in order to prevent chilling. For Hunt Process Clear the application of 1½ cc. will equal 1.23 grams and repressnt a coverage of approximately 208 square feet per gallon on the 8-ounce ointment cans recommended, which have a diameter of approximately 3⅞ inches.

The computed quantity of the curing compound shall be applied on the surface by means of a 2 ml. pipette, or the coverage determined by weighing the specimen as it is being coated. A small camel hair brush previously dipped in the curing compound shall be used to distribute the compound uniformly, or this may be accomplished by tilting the specimen from side to side and causing the compound to flow evenly over the surface.

Note: Where a pipette is not available, an ordinary eye dropper may be used, weighing the specimen before and after application of the compound. In distributing the compound special care should be taken to insure sealing at the line of contact of the mortar and the container. This is particularly important where the container has sloping sides and the mortar has a quick change in thickness. It may be desirable to apply an amorphous wax of approximately 140° melting point to the line of contact in order to be absolutely certain that no moisture will escape along that junction. Anything that is used for the application of the curing compound should be thoroughly cleaned before using a different compound.

Immediately after coating, all specimens shall be weighed. After weighing they shall be placed in an oven or cabinet where the temperature and humidity can be closely controlled. The temperature of the cabinet shall be approximately that of the specimens at the time the specimens are placed therein. The cabinet temperature shall be raised gradually to 100° F. ±5° and kept at this temperature during the period of the test. At intervals of 24 hours after the first weighing the specimens shall again be weighed and the differences between these weighings and the original weighing shall be considered as the loss of water after deducting for the amount of volatile solvent in the curing compound. A humidity of 35% ± 5% shall be maintained in the cabinet during the test period.

Note: It is highly desirable to subject the specimens to uniform temperature (100° F. if possible) in order to obtain the best determination of sealing quality. However, in the field or where a temperature control oven is not available, this is not entirely necessary in order to obtain a comparison, provided all specimens are subjected to identical conditions. The specimens may be left in the building if outdoor temperatures are quite low, or if the outdoor temperatures are above 70°, specimens may be exposed outside where the action of the sun and wind will be present. The test period should be at least five days, or preferably seven days. If the solid contents of the curing solutions are not definitely known, they should be determined by A. S. T. M. method D-154-38 for non-volatile matter. A roughly approximate determination of the volatile content can be made by placing the same amount of curing compound as that applied to the specimen on a cover or plate having the same diameter and weighing this immediately after application of the compound and when it has thoroughly dried. It may be found that the curing compound will not dry as readily on a metal plate as on concrete due to the difference in surface area between the smooth metal and the rough mortar.

At least two specimens shall be coated with each compound and the average of test results taken to determine its sealing quality. The sealing quality shall be based on the weight lost during the test period (water loss) and the amount of water in the specimen at the time the curing compound was applied.

Note: In determining the amount of water in the specimen at the time of the application of the curing compound, allowance should be made for the loss of water between the times of mixing and application, also for any mortar removed during the troweling operation.

Concrete Curing Compounds Manufactured by Hunt Process Company

HUNT PROCESS BLACK—A special blend of selected asphalts with pigment in a petroleum solvent.

HUNT PROCESS CLEAR—A blend of oils and waxes in a petroleum solvent.

HUNT PROCESS CLEAR WHITE PIGMENTED—A special "clear" cure with white pigment for use on thin sections (for example, canal linings) in hot desert climates where temperature control of the concrete is desirable.

HUNT PROCESS COMPANY

operates generally only west of the Mississippi River. Distributors in Dallas, Denver, Honolulu, Kansas City, Mo., Oklahoma City, Phoenix, Portland, Salt Lake City, San Francisco, Seattle, Vancouver, B. C.

Main Office and Plant

7012 Stanford Ave. - Los Angeles, Cal.

As an Integral Curing Agent, "Anti-Hydro" Increases the Strength and Decreases the Permeability of Portland Cement Mixes

When Mr. F. M. Hausling in 1900 started to develop an integral waterproofing for concrete, it was recognized that certain qualities of calcium chloride gauged concrete were desirable. Realizing the importance of the accelerating properties of calcium chloride and the necessity for modifying the salt to protect steel reinforcing, as well as the usefulness of resins and other modifying materials for portland cement, Mr. Hausling, working with Dr. Thomas B. Stillman, started with a calcium chloride solution as a base and built up his present formula. He introduced such additional chemicals, organic and inorganic, which would increase the colloidal possibilities of the cement, and would neutralize and correct the several objectionable properties inherent in a calcium chloride solution per se. The resultant formula was marketed by him under the trade name of "Anti-Hydro," and this formula has remained unchanged for practically forty years.

The success of his work is attested not only by the commercial recognition accorded "Anti-Hydro" but also by tests conducted by many accredited independent laboratories. These include, in part, the laboratories of California Institute of Technology, Case School of Applied Science, Columbia University, E. L. Conwell & Co., Georgia School of Technology, David Kirkaldy & Son, London, England, University of Michigan, Bureau of Standards, H. C. Nutting Co., Raymond G. Osborne, Pittsburgh Testing Laboratories, Smith-Emery Co. and U. S. Engineers.

It is in place to refer to the permeability test on mortars and to permeability and compressive strength tests conducted by the U. S. Bureau of Standards, in which "Anti-Hydro" gauged mixtures were shown to be impermeable while increasing the compressive strength of the concrete at all ages reported. Many calcium chloride products in these same reports were shown to produce more permeable concrete than the standard untreated concrete of the same mix.

There are published herewith microphotographs of crystals from a calcium chloride solution and from "Anti-Hydro." The whole structure of the original calcium chloride crystal is shown to have been altered in the "Anti-Hydro" formula.

The various government laboratory tests and the microphotos are mentioned to emphasize the fact that "Anti-Hydro," unlike many other calcium chloride solutions, is a tried and proven integral waterproofing, developed to bring out more fully the potentialities of portland cement.

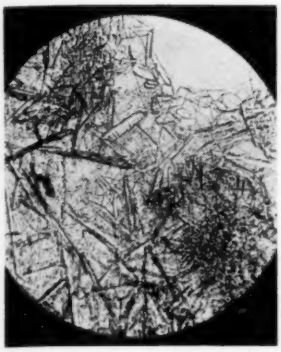
Since density, impermeability and compressive strength are among the measurable criteria of the effectiveness of curing media, test data are presented demonstrating these and other valuable properties imparted to concrete by gauging portland cement mixtures with "Anti-Hydro."

"Anti-Hydro" increases the plasticity and workability of concrete mixtures. Laboratory results reported by the Case School of Applied Science, Dept. of Engineering Research of the University of Michigan, and California Institute of Technology all confirm the increase of slump produced by "Anti-Hydro" while one U. S. Engineers' Laboratory reports the following comparative results on compressive strength and slump.

Test No.	Without "Anti-Hydro"				With 1½ Ga. "Anti-Hydro" Per Cu. Yd.			
	Slump, Inches	Wt. (Lbs.) Per Cu. Ft.	Strength psi. 7 Days	Strength psi. 28 Days	Slump, Inches	Wt. (Lbs.) Per Cu. Ft.	Strength psi. 7 Days	Strength psi. 28 Days
1	2.0	151.1	2440	3560	2.75	150.1	2650	3780
2	2.0	151.1	2420	3820	3.00	150.6	2670	3700
3	2.25	151.1	2440	3540	3.00	151.7	2560	3640
Avg.	2.08	151.1	2433	3640	2.92	150.8	2627	3707

A constant W/C ratio of 6.75 gallons per sack was used throughout the test.

"Anti-Hydro" also increases the tensile strength of portland cement mixtures. This has important bearing upon the bonding qualities of "Anti-Hydro," which were tested by Raymond G. Osborne of Los Angeles, California. "This test was made for the purpose of determining the degree of bond possible between a layer of poured concrete to be used as topping and the old concrete floor slab." Smith-Emery Company of Los Angeles reporting on the shear tests state that "the failure in all cases was in the old concrete. There was no apparent failure of the bond." N. B. The bonding material was a thin portland cement grout mixed with "Anti-Hydro" according to specifications.

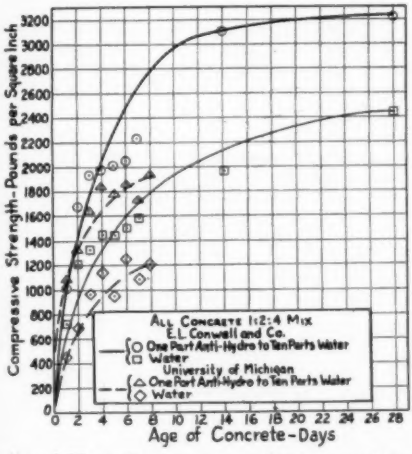


Left, Crystalline Residue From Evaporated Calcium Chloride Solution (28%) 33x

Right, Amorphous Residue From Evaporated "Anti-Hydro" Solution 33x

Increased compressive strength at all ages may be obtained with any standard brand of portland cement through the addition of "Anti-Hydro" to the concrete mix.

The graph shows comparative curves with and without "Anti-Hydro" in the mix as reported by E. L. Conwell & Company, Philadelphia, and Dept. of Engineering Research, University of Michigan, which are representative of innumerable comparative tests obtained in the course of about 40 years of marketing "Anti-Hydro."



"Anti-Hydro" protects reinforcing steel from corrosion. E. L. Conwell & Company, Philadelphia, state in their report of October 9, 1928, "Specimens exposed to the weather up to four weeks showed no sign of corrosion on the steel in mixes containing plain water and 'Anti-Hydro'."

Another function of "Anti-Hydro" is its added protection against frozen concrete and masonry in winter construction, and the earlier use of the concrete. The H. C. Nutting Company of New Jersey reports on a series of cylinders which, during the initial 24 hours, were kept at 15° F. Of 7 without "Anti-Hydro," 3 were frozen. One 1 containing "Anti-Hydro" was found "slightly frozen." At 28 days, the strength of "Anti-Hydro" gauged concrete was 3077 psi., as compared with 2600 psi, for the untreated.

The low w/c ratio and increased workability produced by "Anti-Hydro" lead to concrete impermeable and of low volume change. For almost 40 years the Anti-Hydro Waterproofing Co. has been furthering the production of good uniform concrete not only by the use of its product, but equally as much by furnishing the services of its trained personnel to supervise the mixing and placing of concrete in which "Anti-Hydro" is being used.

Because of the quality product and the field service furnished, it is possible to guarantee the permanence of waterproofness of "Anti-Hydro" gauged concrete and its resistance to abrasion and weathering. The Anti-Hydro Waterproofing Co. can furnish a five year maintenance guarantee on supervised jobs.

The data given above are merely suggestive of what may be expected of "Anti-Hydro" as an internal curing agent for durable concrete. Manufacturer's field service, when desired, is available to assure the proper mixing and placing of the concrete.

Stock available in all principal cities

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FOR PERMANENT CONCRETE
HARDENS WATERPROOFS

265 Badger Ave.
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THE CALCIUM CHLORIDE ASSOCIATION, 4145 PENOBSCOT BLDG., DETROIT

High Early Strength Concrete Specifications for Architects—Design Engineers—Construction Engineers

For high early strength and, in cold-weather concreting to speed up construction, specify flake calcium chloride in concrete. This will permit earlier finishing and earlier removal of forms and will save time and materials in construction of footings, floors, walls and columns when used at the following schedule.

General recommendations are to use 2 lb. of calcium chloride per sack of cement at temperatures below 80 F. In warmer weather, 1 to 1½ lb. will give high early strength and the lesser amounts are used to prevent too rapid hardening.

At lower temperatures (50°–32°F) the use of calcium chloride sufficiently accelerates the rate of hardening to permit finishing and maintain construction progress on summer schedules.

When temperatures fall to from 32 to 25 F., the use of 3 lb. and, at temperatures below 25 F., the use of 4 lb. of flake calcium chloride per sack of cement will further speed up the hardening, and with proper heating and protection, will most rapidly get concrete out of danger from freezing or frost damage.

Concrete mixed on project site:

Flake calcium chloride shall be added to each batch at the quantities prescribed for the lowest temperature anticipated between placement and six hours following placement of the concrete. A measure (dipper or pail) shall be marked off to indicate the proper quantity of calcium chloride required, based on the cement content of each batch. (A pint is a pound, so a full quart of flake calcium chloride for each sack of cement meets the regular specification.) The calcium chloride is added to the aggregates immediately before being discharged into the mixer drum or added directly to the mixer drum after the water and aggregates have been entered. The mixing shall be continued for a minimum of one minute.

Transit-Mix Concrete:

Flake calcium chloride can be added to mixer drum upon delivery to the project site as described in the preceding section. The mixing shall then continue for a minimum of one minute before discharging.

If mixing water is added enroute to the project site, calcium chloride may be added in solution of desired concentration previously dissolved in the mixing water. Flake calcium chloride may be added at the plant provided the batch is to be mixed immediately for at least one minute and the concrete is to be discharged at the project within one hour.

Central Plant Mixed Concrete:

Flake calcium chloride may be added at the plant provided the concrete is discharged at the project site within one hour.

Masonry Mortar:

Flake calcium chloride shall be added to the mortar at the rate of 2 lbs. per bag of portland cement, or 1 lb. per bag of prepared mortar cement, whenever an early hardening product is desired. The calcium chloride shall be dissolved in water at the proportions of 4 lb. of calcium chloride to 1 gallon of water. Each quart of solution will then contain 1 lb. of calcium chloride and 1 quart of solution should be added for each bag of mortar cement or 2 quarts for each bag of portland cement.

Detailed data on the effects of calcium chloride in attaining high early strength and for cold weather concreting as investigated under research studies at the National Bureau of Standards are reported in the Highway Research Board Proceedings (Paul Rapp—1934 and Jason C. Yates—1941) and are summarized in Bulletin No. 28 on Early Strength Concrete. Copies available on request to the Calcium Chloride Assn. —4145 Penobscot Bldg., Detroit, Mich.

MEMBRANE CURING

Its history, development, and the advantages that accrue by its use.

(Q) ***What is Membrane Curing?***

(A) Membrane curing is the application to newly placed exposed concrete surfaces of an impervious continuous coating designed to restrict evaporation of the mixing water.

(Q) ***Is Membrane Curing new?***

(A) No. As early as 1921 German Patent No. 337,134 was granted to Paul Lechler of Stuttgart. An American Patent No. 1,684,671 was issued in 1928 (since held invalid).

(Q) ***What types of Membrane were first used?***

(A) Bituminous materials (coal-tar pitch cutback, asphalt cutback and asphalt emulsions).

(Q) ***Were they satisfactory?***

(A) Investigations conducted in various laboratories, and corroborated by use, demonstrated that certain of the coal-tar and asphalt cutback materials were quite effective for the retention of concrete mixing water and, in some respects, a satisfactory substitute for water curing.[1]

(Q) ***What objections were raised to their use?***

(A) Two principal ones: undesirable appearance and their tendency to promote excessive cracking in concrete as a result of their high heat-absorption properties when exposed to sunlight. This resulted in their use being restricted to unexposed surfaces and on structures which are later backfilled.[2]

(Q) ***What were the early "Clear" Membranes?***

(A) Two types—sold under a so-called "Impervious Membrane Curing License."

(Q) ***Were they identical?***

(A) No—one type, marketed in the West, was a compound of a paraffine-vegetable oil base cut with petroleum solvent. The second, marketed in the East, was a compound of a gum rosin base cut with petroleum solvent.

[1] "Solving the Problem of Concrete Curing on the Colorado Aqueduct," by L. H. Tuthill, Western Construction News, August, 1935.

[2] "Concrete Curing by the Use of Membranous Coatings," a paper presented at Highway Engineering Conference, University of Colorado, January 20, 1939, by G. E. Burnett, Junior Engineer, U. S. Bureau of Reclamation.

(Q) ***What were the advantages claimed for them?***

(A) That in addition to providing satisfactory retention of moisture, excessive heat absorption was eliminated and concrete was maintained in its natural appearance.

(Q) ***Did their performance justify these claims?***

(A) No. Laboratory tests and field reports indicated that they were ineffective because they did not satisfactorily retain the proper amount of moisture in concrete to provide adequate curing. [2] [3] [4]

In addition, there were many instances where the compounds imparted a blotched discoloration to concrete, which qualified authorities ascribe to the *reaction* between the calcium hydroxide present in the concrete and the free fatty acids of the compounds, resulting in the formation of calcium salts of the fatty acids, with ensuing discoloration and softening of the surface.

(Q) ***Is this reaction harmful?***

(A) Yes. "Vegetable and animal oils and fats are natural products composed of glycerides and esters, but they also contain some of the corresponding free fatty acids and alcohols. When such oils are brought into contact with concrete they react with the calcium hydroxide present in the hardened concrete, forming a calcium salt of the fatty acid.

"The extent of the oil attack is dependent on the ease with which it can penetrate the concrete. Thus the viscosity of the oil, as well as the porosity of the concrete, is an important factor. Oils which have been exposed and have accumulated moisture and undergone oxidation become more active in attack." [5]

(Q) ***What led to the development of efficient curing compounds?***

(A) Research—instigated by us in 1938 at the request of Engineers and Concrete Technicians, who, because of difficulties experienced with water curing, moisture retentive fabrics, and reactive type multiple-coat curing compounds, called upon us to develop a ONE COAT clear curing compound that would conform to specified tenets.

(Q) ***What were these tenets?***

(A) Immediately and effectively seal the surface against surface dehydration, and provide maximum retention of the original mixing water within the concrete.

Be chemically inert to any constituents of concrete.

Produce concrete having compressive and flexural strengths equal to 14-day water curing.

Produce concrete surfaces having erosion and abrasion resistance equal to 14-day water curing.

(Q) ***What was the basis of this research?***

(A) To evaluate available raw materials by accurately measuring their *resistance to moisture passage and their chemical inertness.*

(Q) ***How extensive was this research?***

(A) Our investigations included practically every type of film-forming material known to possess high impedance to moisture passage. These comprised over three hundred materials and compounds, including practically all commercially available oils, waxes, rosins, resins, bituminous compounds and chemical products.

[3] "Tests of Concrete Curing Materials," by F. J. Jackson and W. F. Kellermann—ACI Journal, June, 1939 and September, 1939.

[4] "Concrete Curing Compounds," by H. S. Meissner and S. E. Smith—ACI Journal, May-June, 1938, and September, 1938.

[5] "The Chemistry of Cement and Concrete," by F. M. Lea and C. H. Desch.

MEMBRANE CURING

(Q) ***What did this research disclose?***

(A) That, unfortunately, most resins and all of the drying oils which could logically form a basis of a semi-permeable film, either failed to achieve satisfactory moisture retention, or reacted with the concrete to form calcium soaps.

(Q) ***Did any material possess all of the desired properties?***

(A) Yes—one type of resin (a polymerized ring compound, made under a patented process) and later a similar type of approximately the same theoretical structure, both of which offered the highest resistance to moisture passage and were chemically inert.

(Q) ***What was their efficiency in percent moisture retained?***

(A) Approximately 85 percent.

(Q) ***Did we consider this the maximum obtainable?***

(A) No—because we discovered during our research that amorphous water-repellent aquaphobes, when colloidally dispersed in the solution, resulted in added efficiency. This is *the principle of stratification* for which a patent grant has been requested.

(Q) ***What were the physical characteristics of the resulting Membrane?***

(A) The curing film, or membrane, formed upon evaporation of the solvent from the sprayed coating, had a moisture retention efficiency of over 95% and was comparable to a laminated structure in that the membrane formed consisted of:

(1) A lower layer of a highly impermeable, non-reactive, inert polymer (or vehicle) in contact with the concrete.

(2) An upper stratum of approximately parallel layers of dispersed, amorphous, water-repellent aquaphobes ("leafing" much in the same manner as aluminum powder), with a thin cement of vehicle between each layer and particle.

The arrangement of the aquaphobes in this stratum is such that it forces the water to follow a much longer path around and between these innumerable water-repellent bodies if it is to escape through and reach the surface of the membrane. (See Figure 1). This is equivalent to multiplying the film thickness several times and accounts for the remarkably high impedance of this type of membrane to the passage of water (in either the liquid or vapor phase), when applied in just ONE coat.

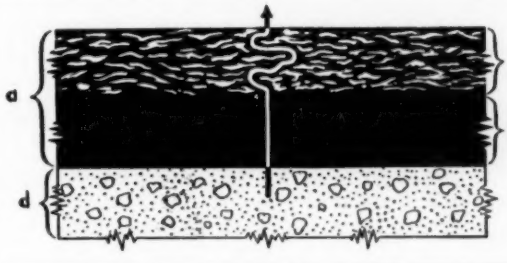

Figure 1

a—Complete curing film.

b—Lower layer of inert polymer.

c—Upper layers of dispersed aquaphobes.

d—Concrete.

Arrow shows path water must follow if it is to escape.

(Q) ***Was this considered a new and positive method of curing?***

(A) Yes—as a result of this comprehensive investigation we announced production of AQUASTATIC CLEAR 1-C, the first factually effective, non-reactive, colorless concrete curing compound that produced a film of near-zero porosity in one application.

(Q) ***Were these findings confirmed?***

(A) Yes—confirmed and being continually reconfirmed by numerous technicians and concrete research laboratories of nationally recognized importance.

(Q) ***Have laboratory results been confirmed in field practice?***

(A) Yes—as evidenced by the fact that numerous specifications of governmental agencies specify AQUASTATIC by name, and by the further fact that all of the concrete on hundreds of the more important war construction projects has been and is being cured with AQUASTATIC. These include locks, dams, drydocks, airbases, ordnance plants, access roads, etc.

(Q) ***Have these results been given official recognition?***

(A) Yes—in the "Tentative Specification for Curing," inclosure to Circular Letter 2156, Construction Division No. 558, dated November 23, 1942, War Department—Office of the Chief of Engineers, it is stated:

> "*Curing Compounds.* Curing compounds of satisfactory composition and characteristics may be used. Moisture retention, viscosity, ease of application, rate of hardening, texture of the film, absence of shrinkage cracks or holes in the membrane, percentage of non-volatile solids, adhesion to both vertical and horizontal surfaces, resistance to rain during the hardening process, reaction with the concrete ingredients, and other characteristics of the compound will be considered. A compound will be approved if, in the opinion of the Contracting Officer, laboratory tests and field performance demonstrate that the above characteristics are equal to those secured with Klearcure (No. 70), SealKure, Tru-Cure (No. FX-199 or 203), or AQUASTATIC (1-C, 1-FMST, or Black) as submitted for test at the Central Concrete Laboratory, Mount Vernon, N. Y., in June, 1942."

Note: Of particular interest and significance is the fact that these other resin base compounds mentioned are formulated on the AQUASTATIC principle of chemical inertness and stratification, which is apparently the only method of formulating a membrane curing compound to meet the high standards now in effect.

(Q) ***Will this specification preclude the use of inefficient compounds?***

(A) Yes—as far as the Corps of Engineers is concerned. Unfortunately, however, many so-called curing compounds are currently being offered to take advantage of the obvious benefits which the method brings, without the background of research and field application.

(Q) ***Does AQUASTATIC meet other specifications?***

(A) Yes. AQUASTATIC is admissible under and surpasses the requirements of any specification with which we are familiar. This includes the recommended specification of the Highway Research Board, calling for 85% retention, and exceeds the 90% retention mentioned in the Board's footnote to *Par. a* of their specification.

(Q) ***What are AQUASTATIC'S specific benefits to the war construction program?***

(A) The use of AQUASTATIC reduces to one operation the multiplicity of operations required for any other method, and in so doing, it—

1. requires but one inspection;
2. eliminates the burden and expense of exacting and prolonged supervision;
3. keeps work area dry and prevents all delays incident to sloppy working conditions;
4. avoids the possibility of decompaction through wet subgrade;
5. effects definite economies in cost and adds materially to the speed of construction;
6. produces concrete of high strength with a maximum performance expectancy.

MEMBRANE CURING

(Q) ***Can subsequent coatings be applied over AQUASTATIC?***

(A) Yes—because AQUASTATIC is inert and does not form soap or contain wax; it provides a film which makes a perfect bond for subsequent coatings, such as camouflage, reflectorized stripes, or the additional waterproofing called for in some specifications.

(Q) ***How is an engineer to know which curing compound to specify?***

(A) By a careful scrutiny of the chemical and physical characteristics, as well as the performance record of any material offered at standard coverage (200 square feet per gallon) and by requiring vendor to furnish test results of impartial laboratories substantiating his claims of moisture retention, chemical inertness and other properties.

(Q) ***What about the future of Membrane Curing?***

(A) We believe that, if it is properly done, the advantages which make membrane curing now essential will be equally valuable in postwar problems.

We are jealous of the good name of Membrane Curing. We expect to be manufacturing AQUASTATIC long after the war, and we think the advantages in the method are such that you should be just as anxious to have it available. This can only be done if *efficient* membrane curing compounds are *properly* used in these abnormal times. AQUASTATIC Concrete Curing Compounds, of proper efficiency, completely unreactive, are available in unlimited quantities and prompt shipments can be made.

Aquastatic
CONCRETE CURING COMPOUND

SOLVENTS AND PLASTICS COMPANY
8032 Forsythe Blvd. • St. Louis, Missouri
Laboratories: 35th and Tyler Sts. • Louisville, Kentucky

NATIVE HABITAT:

U.S.A.

America's "Trojan Horse" was not stationed within our walls by the enemy. It was our own unique creation—*an offspring of democracy itself* Reared on the concepts of self-sufficiency and the many freedoms, nurtured along by personal ambition and contentment, this false idol gradually took shape. Its name is COMPLACENCY—its paralytic effect on the people of America has made it the most dangerous of Axis agents. How to combat it? First by recognizing it for what it is—*a monstrous stumbling block to the Allied cause.* Second—by putting aside every personal gain but that of Victory, by personally contributing to the war effort. Wake up America! It's time to stop living off the luxury of democracy. Time to take steps to preserve our *native habitat!*

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Second Annual

TECHNICAL PROGRESS SECTION

FOR COMMERCIAL PRODUCERS

The Preceding 59 Pages

A part of PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE Vol. 39

JOURNAL
of the
AMERICAN CONCRETE INSTITUTE

Vol. 14 No. 5 7400 SECOND BOULEVARD, DETROIT, MICHIGAN April 1943

Looking Backward and Forward*

By MORTON O. WITHEY†
President American Concrete Institute

PRESIDENT DAVIS' TELEGRAM TO CONVENTION

Regret inability to be with you who are present at ACI convention. When war is over we will have convention surpassing in quality and size anything yet dreamed. Meantime Institute and its members will continue to give all to winning the war, not forgetting that peacetime potentialities of concrete loom larger than ever before and that when peace comes it is up to us to see that these potentialities are realized.

—RAYMOND E. DAVIS

When notified a few days ago that President Davis was unable to attend this token convention, I reviewed the addresses of the seven presidents last past, in order to ascertain their thoughts regarding the progress and ways of improving the Institute. Following this review, I hastily catalogued the contents of the JOURNAL for 1942, to see how nearly our course had followed the general trend desired by these officials. I shall state the results of this comparison, report briefly upon the present status of the Institute, and make a few suggestions concerning the work of the coming year.

Consideration of the remarks of the past presidents showed that the central theme in the mind of each was the proper fulfillment of the objectives of the founders stated in the Charter.

The four earlier presidents of this group of seven were much concerned with the lack of published material of interest to the man on the job, the craftsman, and the user of concrete. Some voiced alarm at the preponderance of academic and technical papers which appeared in the JOURNAL, and were afraid that the Institute would become another pseudo-scientific society whose proceedings would fail to appeal to most practitioners and manufacturers.

*Presented at the Institute's 39th Annual Convention, Chicago, February 17, 1943, by the Vice-president and President-Elect, Prof. Morton O. Withey. Annually on such occasions the "President's Address" is by the retiring president. Professor Withey's remarks were made in the absence of the retiring President, Prof. Raymond E. Davis, who, convalescing at his home, Berkeley, Calif., after a long illness, sent the telegram which his successor read to the convention.

†Professor of Mechanics, The University of Wisconsin, Madison, Wis.

Important policies enunciated by our past leaders for the creation of a successful association are:

1. The Institute should be a general clearing house for information on concrete.

2. This information should be published in such form that the essential facts will be understood not only by technicians but also by those engaged in construction and the manufacture of products, by the craftsmen, and the users of concrete.

3. The aim should always be more and better papers and more and better standards. The presentation of many short papers on diverse subjects within the field should be encouraged.

4. The assignment of members to committees should be based on the interest of members in the assignment, but a wholesome freedom from a prejudiced viewpoint should also be maintained.

5. Committees should show progress on an assignment or resign.

6. The lives of standards should be limited to a short period, three to five years, after which the standards should be discarded or revised.

7. Conventions should be made attractive to *all* interested in concrete, whether that interest be in design, construction, inspection, products manufacture, craftsmanship or other field of application or use of concrete.

The foregoing are excellent objectives, but their attainment is only possible through the whole-hearted cooperation of the membership and the realization by each member of his individual responsibility to the Institute.

During the trying year 1942, 25 papers and reports were published in the JOURNAL, many of them were followed by well-considered discussions. Of these papers, I listed five on design, eight on construction, one on inspection, six on material, and four committee reports. The Job Problems and Practice Section carried 42 articles, 13 on design, 16 on construction, 4 on inspection, and 6 on materials. Ninety-nine reviews were included in the JOURNAL during the year. The Technical Progress issue began the novel attempt on the part of the Institute to encourage advertising based upon the results of technical information. Twenty-six were represented in the 44 pages devoted to this successful venture.

Thirty technical committees are engaged as follows: Research, 1; general properties, 5; engineering design, 8; construction, 10; shop manufacture, 5; repair and maintenance, 1. In addition, the Institute has representatives on fifteen joint committees drawing memberships from several interested technical societies.

Membership in the Institute on June 30, 1921, numbered 688; in 1924 1289; in 1929, 2616. In 1934 the number, due to the effect of the financial depression, had dropped to 1154, and then increased steadily until we now have 1883, an increase of 125 during the past year.

A brief review of our financial record shows that our expenditures in 1929 approximated $34,000. Our maximum surplus was $20,464 in 1930; our minimum annual expenditure was $17,414 in 1934; and our minimum surplus was about $7,900 in 1935. Today, as a result of some remunerative publications and some nice advertising, our surplus is $19,667.39. Expenditures for the past year were $31,581.*

Considering the evidence just submitted and also the difficulties besetting the nation and its citizens, I think you will agree with me that the Institute has completed a very successful year. A large share of the credit for this gratifying condition is due to our vigorous Secretary and his busy staff. His office has been most effectively aided by the members of the Advisory, Publications, and Standards Committees, and the cooperation of all authors of papers and committee reports.

During the coming year it would seem to me highly desirable for the Institute to strive through its committees and publications to encourage all worth-while efforts which are in any way related to the war effort. More specifically, let us accelerate our efforts to publish committee reports, interpret existing research data, furnish design methods and good practices, using as a basis the best information now at hand. Reports from committees dealing with proper bond stresses, the resistance of concrete to abrasion and erosion, the cold weather protection of concrete, behavior of rigid frame bridges, methods of detailing continuous beams and frames, methods of estimating safe loads for bridges, the design of pre-stressed reinforced concrete, winter concreting methods, recommendations for placing concrete by vibration, recommended practice for the manufacture of concrete masonry units, pre-cast floor systems for houses, methods of applying mortar by pneumatic pressure would be decidedly helpful to the war effort.

Realizing that many members of the Institute and its committees are already serving in war emergency efforts, the decision as to which effort is the more important will necessarily rest with the individual.

Having given priority to the publication and dissemination of information which is directly related to the war effort, let us also keep alive the spirit of research and extend the boundaries of knowledge within our field by publishing the results of new experiments, new practices, and

*For the fiscal year ended June 30, 1942.

new methods of design as rapidly as possible. By so doing we shall materially "shorten the lag between research and practice."

At present money is relatively plentiful for memberships in technical societies. Members should zealously urge interested persons of proper qualifications to join the Institute. Manufacturers and firms on our field of endeavor should receive cordial invitations to become Contributing Members.

During the past five years the sessions of our conventions devoted to the reports of Research Committee 115 have been well attended and stimulating. It is suggested similar sessions be considered in other fields, prehaps one session might be devoted to construction problems, another to problems in design.

A part of PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE Vol. 39

JOURNAL
of the
AMERICAN CONCRETE INSTITUTE

Vol. 14 No. 5 7400 SECOND BOULEVARD, DETROIT, MICHIGAN April 1943

Long Span Concrete Roof Construction*

By C. H. MAYER†

SYNOPSIS

The paper shows the reinforced concrete roof design used by the author during the last 25 years for economy and speed. It is of special interest in its minimum use of critical materials in these times. The notable feature of the design is a simple Howe truss whose top chord members support a continuous monitor for light and ventilation.

Since the War Production Board's ruling limiting the use of structural steel for building purposes, engineers and architects have looked more and more toward concrete and wood to solve their needs for construction.

The use of wood trusses and built up wood girders for long span construction in all types of buildings has greatly increased and found to be satisfactory with certain limitations. Such structures as foundries, forge shops, heat treatment buildings, etc., must necessarily be of more fire resistive material.

Several types of long span concrete buildings have been developed—the primary object in the design of these buildings being the reduction to a minimum of the use of vital materials. Concrete arch construction has been resorted to with great economy. Arch construction, however, is not adaptable to roofs requiring skylights and monitors because complicated details of construction result in loss of economy and waste of reinforcing steel.

The following is a description of a type of long span reinforced concrete roof designed and successfully used by the writer during the last twenty-five years.

The design consists of a simple Howe truss in which the top chord members of the truss extend beyond the apex of the roof to support a continuous monitor admitting required light and ventilation.

*Received by the Institute Dec. 30, 1942.
†Schmidt, Garden & Erikson, Architects, Chicago.

Fig. 1 (above)—Building, of the type described, under construction. It has a crane rail span of 80 ft. The framework in the background is readily assembled and knocked down for reuse

Fig. 2 (below)—Building under construction has three spans of 60 ft. each.

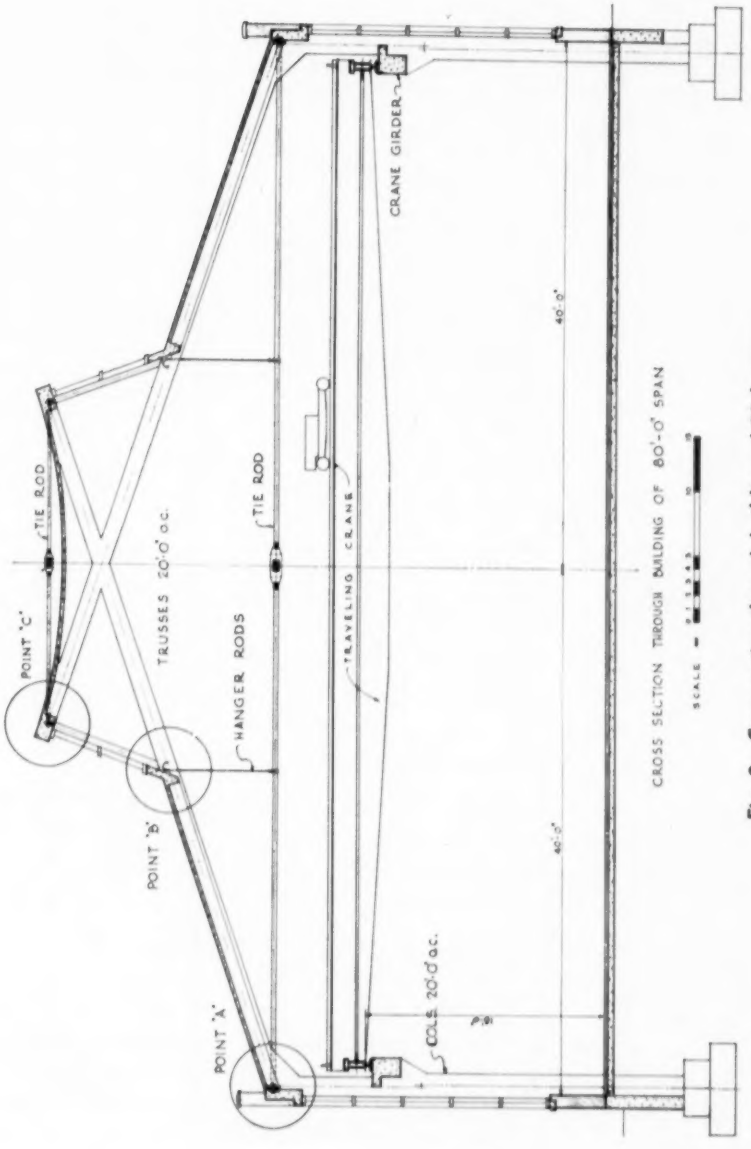

Fig. 3—Cross section through building of 80 ft. span

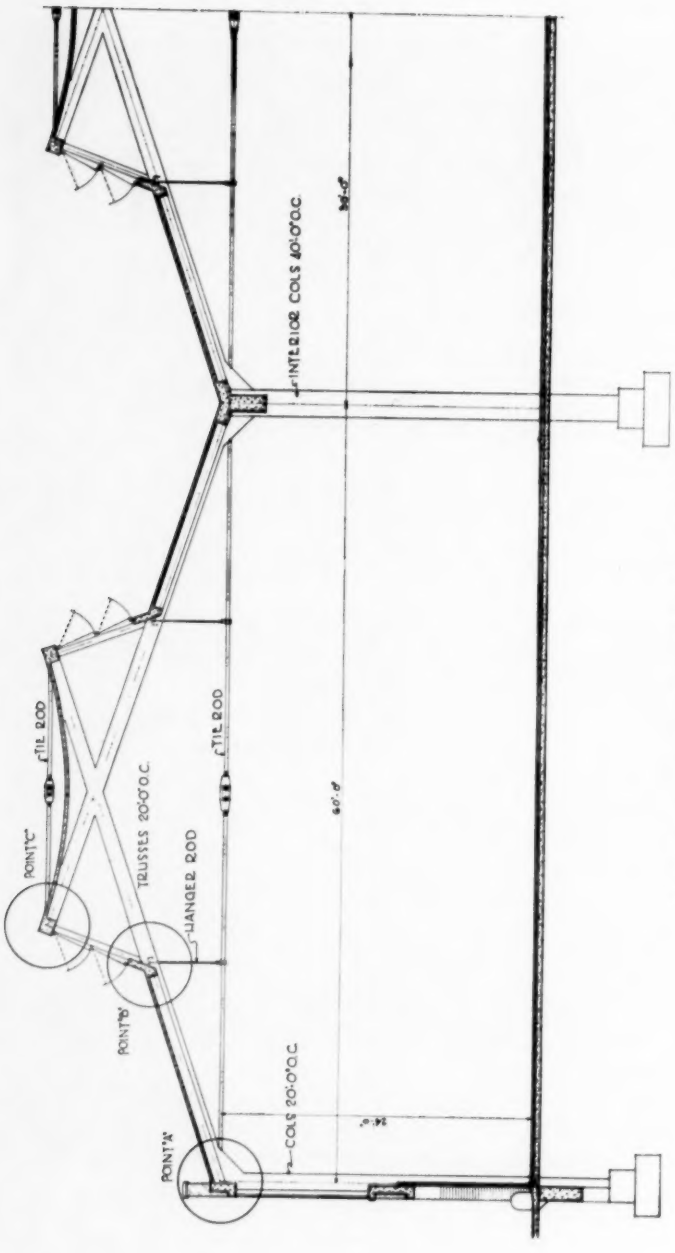

Fig. 4—Cross section through building of triple 60-ft. span

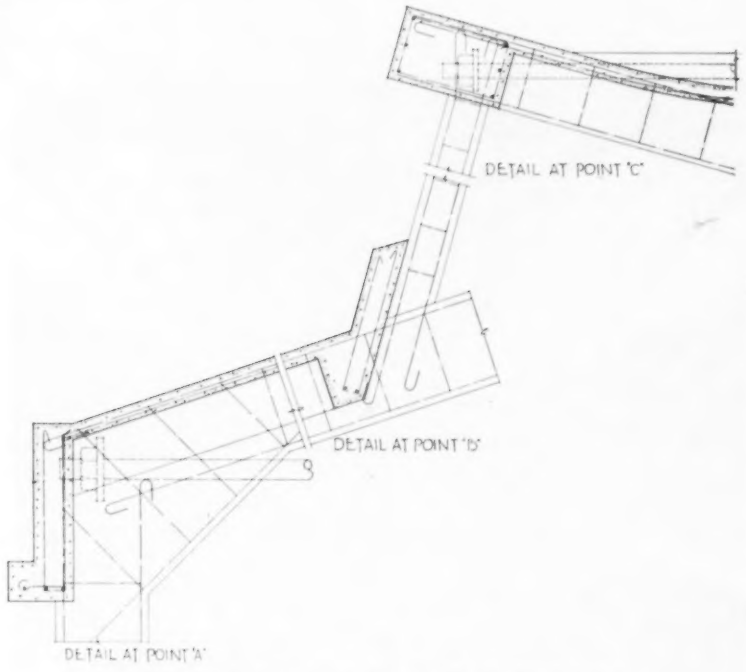

Fig. 5—Details of points A, B and C of Fig. 3 and 4

The roof of the monitor is formed on the curve of a catenary eliminating all bending stresses in the roof slab for normal loading. The reinforcing steel hence must carry this load in direct tension and the thickness of the concrete shell encasing the rods may be reduced to a minimum.

The supporting rods are hooked into continuous longitudinal beams on each side of the monitor designed and reinforced in the direction of the strain from the catenary. The beams are framed into and held by the extended top chord members of the truss.

The main roof slab of the building consists of an ordinary one way reinforced slab supported on rafters 6 ft. 8 in. o.c. The rafters are framed into the spandrel beam on the one side and into a continuous beam formed to include the sill of the monitor sash on the other side, the latter being supported on the top chord member of the trusses. To eliminate excessive flexural stresses due to load concentrations on the truss members, the reactions from these main roof supporting beams are carried through an inclined tie to the end of the cantilever above, there combined with the reaction from the monitor roof and the resultant carried through a horizontal tension rod across the monitor and there counterbalanced by equal loads acting in opposite direction.

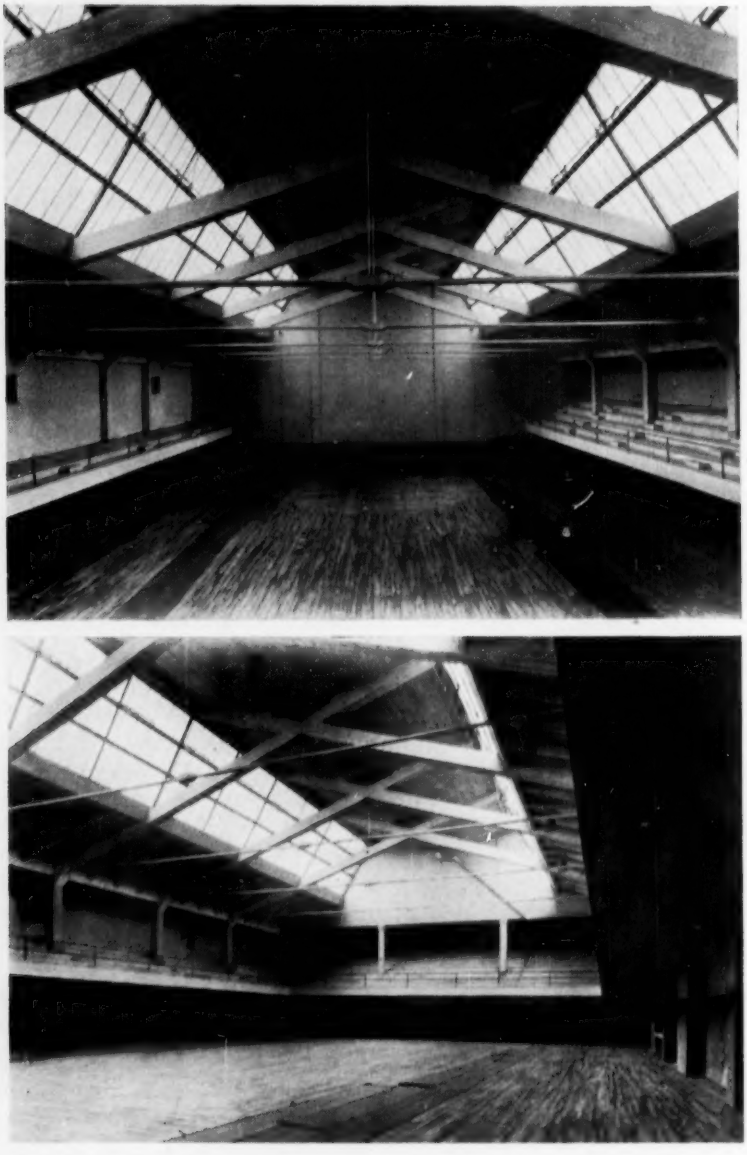

Fig. 6 and 7—Buildings in use—of types similar to those described

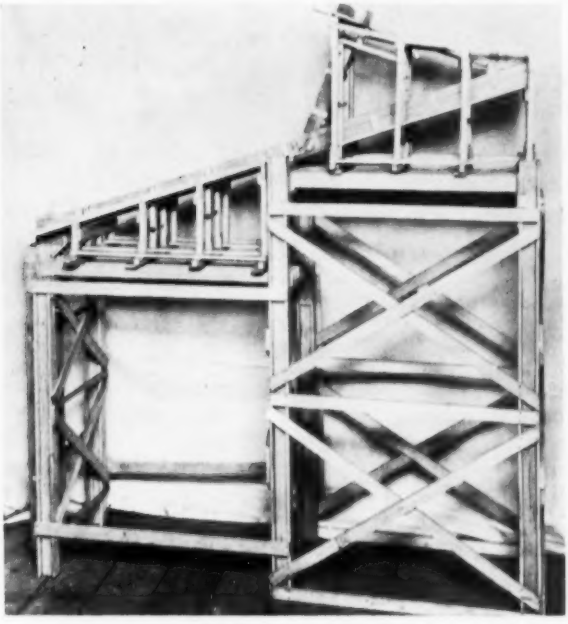

Fig. 8—Model of form work adopted for simplicity of erection and speedy knock-down for reuse

The bottom chord of the truss consists of an ordinary tie rod embedded into the building columns with proper plates to transfer the roof loads into the rods. Turnbuckle is provided at the center and sag rods as may be needed.

Haunches are formed at the intersection of the columns and truss members to resist the wind pressure.

Fig. 1 shows a building of this type under construction with a crane rail span of 80 ft. The framework shown in the background is easily assembled and knocked down for reusing.

Fig. 2 shows a building partially completed consisting of three spans each 60 ft. with interior columns 40 ft. o.c., alternate trusses carried on concrete girders framed into these columns.

In the two buildings shown on Fig. 1 and 2, two channels with turnbuckle at center screwed into threaded forging, were substituted for the usual tie rods, as those rods were not immediately available.

Fig. 3, 4 and 5 show a cross section and details of the design of these buildings.

Fig. 6 and 7 show completed structures of similar type of construction now in use.

Fig. 8 shows a model of the form work adopted for its simplicity of erection and speedy knock down for reusing.

The following comparison of two of a group of buildings recently constructed for a defense plant will show clearly several items of interest:

	Forge Shop	Die & Maintenance Bldg.
Construction	Structural Steel	Reinforced concrete of type described herein
Cost	$5.13 per sq. ft.	$4.60 per sq. ft.
Required Steel	25 lb. per sq. ft.	4.75 lb. per sq. ft.
Insurance Rate	8 cents per $100	4.5 cents per $100

Due to the uncertainty and delay in the delivery of structural steel the die and maintenance building was entirely constructed and enclosed before the erection of structural steel for the forge shop commenced, hence the old complaint of the slowness of the erection of concrete structures for the present at least may be eliminated.

A fact repeatedly demonstrated over a long period of years is also that the cost of maintenance of the concrete structure is only a fraction of that of unprotected structural steel.

Discussion of this paper should reach the ACI Secretary, in triplicate by July 1, for publication in the November Supplement, concluding Proceedings Volume 39.

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Impact Resistance of Reinforced Concrete Slabs*

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Member American Concrete Institute

SYNOPSIS

Impact tests of 15 reinforced concrete slabs indicated that the use of supplementary reinforcement in the form of a series of overlapping spiral coils increased their impact resistance by 1½ to approximately 2½ times with energy loads varying from 1400 ft. lb. to 5000 ft. lb. The relative impact resistance with respect to weight per sq. ft. of slab and weight of reinforcement in a sq. ft. of slab was determined for various patterns of the bar reinforcement in the slab and for various thicknesses of slab. The tests were made at the National Bureau of Standards for the Maritime Commission.

1—INTRODUCTION

The current restrictions on the use of steel have again aroused interest in the utilization of reinforced concrete for constructing seagoing cargo ships and barges. One of the matters of chief concern, obviously, is the resistance of the relatively thin shell of concrete to impact loads concentrated over a small area. This paper is a report of the results of tests conducted at the National Bureau of Standards for the Maritime Commission on the resistance to large impact loads of reinforced-concrete slabs, the designs of which were similar to those proposed for the hulls of reinforced concrete ships.

The primary purpose of the tests was to compare the behavior, under impact load, of slabs with and without the supplementary reinforcement in the form of overlapping helices of large diameter wire, commonly known as spirals. Comparisons were also made between slabs differing in thickness as well as in arrangement and in quantity of reinforcement. A total of 15 slabs was tested, of which all but two were constructed of Haydite‡ aggregate concrete. The specimens were subjected to successive impact loads increasing in intensity until failure occurred, except for two slabs which were subjected to a single impact blow of sufficient

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‡Haydite is an artificial light weight aggregate produced by burning shale to the point of incipient fusion.

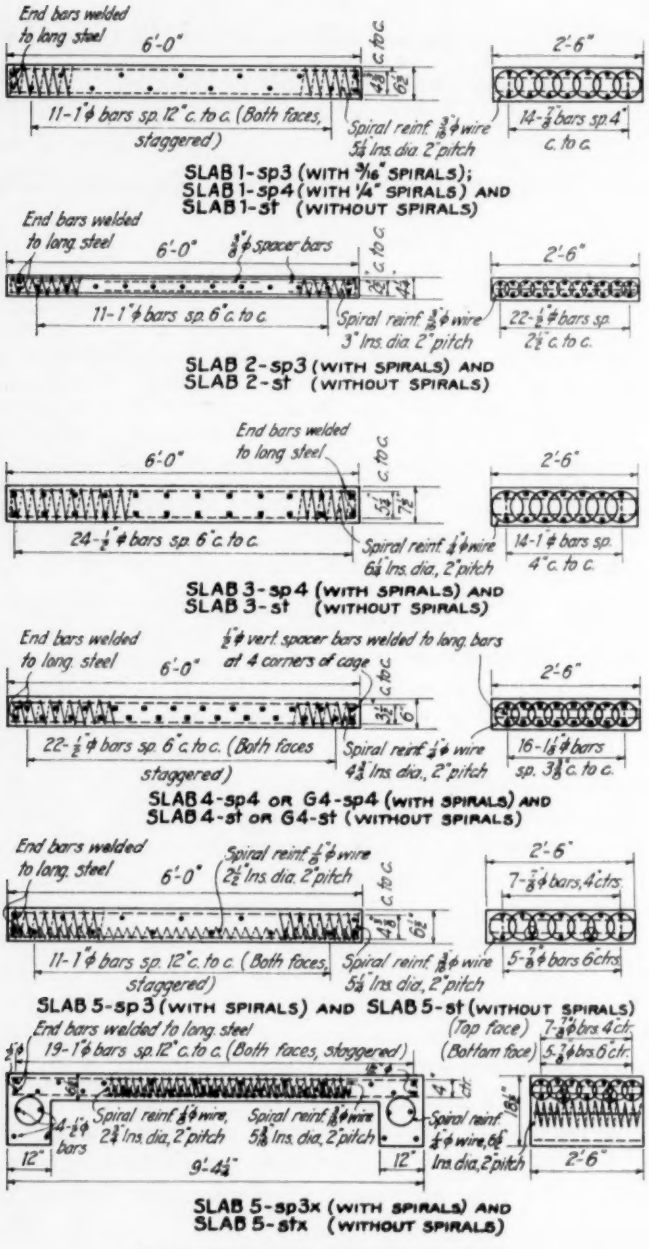
End bars welded to long steel
6'-0"
c. to c.
2'-6"
11-1"ϕ bars sp. 12" c. to c. (Both faces, staggered)
Spiral reinf. 3/16"ϕ wire 5½" Ins. dia. 2" pitch
14-7/8" bars sp. 4" c. to c.
SLAB 1-sp3 (WITH 3/16" SPIRALS);
SLAB 1-sp4 (WITH ¼" SPIRALS) AND
SLAB 1-st (WITHOUT SPIRALS)
End bars welded to long steel
6'-0"
⅜"ϕ spacer bars
c. to c.
2'-6"
11-1"ϕ bars sp. 6" c. to c.
Spiral reinf. 3/16"ϕ wire 3" Ins. dia. 2" pitch
22-½"ϕ bars sp. 2½" c. to c.
SLAB 2-sp3 (WITH SPIRALS) AND
SLAB 2-st (WITHOUT SPIRALS)
End bars welded to long steel
6'-0"
c. to c.
2'-6"
24-½"ϕ bars sp. 6" c. to c.
Spiral reinf. ¼"ϕ wire 6½" Ins. dia., 2" pitch
14-1"ϕ bars sp. 4" c. to c.
SLAB 3-sp4 (WITH SPIRALS) AND
SLAB 3-st (WITHOUT SPIRALS)
End bars welded to long steel
½"ϕ vert. spacer bars welded to long. bars at 4 corners of cage
6'-0"
c. to c.
2'-6"
22-½"ϕ bars sp. 6" c. to c. (Both faces staggered)
Spiral reinf. ¼"ϕ wire 4¾" Ins. dia., 2" pitch
16-1⅛"ϕ bars sp. 3⅜" c. to c.
SLAB 4-sp4 OR G4-sp4 (WITH SPIRALS) AND
SLAB 4-st OR G4-st (WITHOUT SPIRALS)
End bars welded to long. steel
Spiral reinf. ¼"ϕ wire 2½" Ins. dia. 2" pitch
6'-0"
c. to c.
2'-6"
7-⅞"ϕ bars, 4" ctrs.
11-1"ϕ bars sp. 12" c. to c. (Both faces, staggered)
Spiral reinf. 3/16"ϕ wire 5⅛" Ins. dia., 2" pitch
5-⅞"ϕ bars 6" ctrs.
SLAB 5-sp3 (WITH SPIRALS) AND SLAB 5-st (WITHOUT SPIRALS)
End bars welded to long. steel
19-1"ϕ bars sp. 12" c. to c. (Both faces, staggered)
(Top face) 7-⅞"ϕ brs. 4" ctr.
(Bottom face) 5-⅞"ϕ brs. 6" ctr.
4-1"ϕ bars
Spiral reinf. ¼"ϕ wire, 2¾" Ins. dia., 2" pitch
Spiral reinf. 3/16"ϕ wire 5⅛" Ins. dia., 2" pitch
Spiral reinf. ¼"ϕ wire, 6½" Ins. dia., 2" pitch
12"
9'-4½"
12"
2'-6"
SLAB 5-sp3x (WITH SPIRALS) AND
SLAB 5-stx (WITHOUT SPIRALS)

Fig. 1—Details of impact test specimens

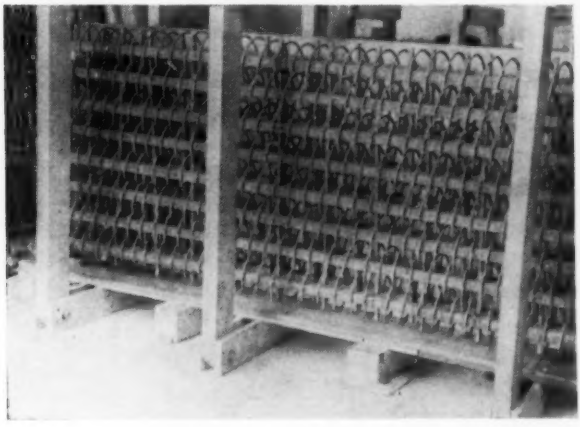

Fig. 2—View of spiral reinforcement

magnitude to cause immediate failure. In general, the energy loads varied from approximately 600 ft. lb. to 10,000 ft. lb., with computed velocities from 5½ ft. per sec. to 22½ ft. per sec., respectively, at point of impact.

The tests were planned by the Vacuum Concrete Corporation and the test specimens constructed by their technical staff in the laboratories of the National Bureau of Standards.

2—DESCRIPTION OF TESTS

1. Description of test specimens

Design details of the various slabs are shown in Fig. 1, and a view of the reinforcement for one of the test specimens in Fig. 2.

All of the specimens, except the two which were subjected to a single, destructive impact blow, were 6 ft. long and 30 in. wide, and varied from 4¼ in. to 7½ in. thick. Each face of all slabs was reinforced with a grid of reinforcing bars, the size and spacing of the bars depending upon the design, and the two grids formed a unit by being tied to each other with short spacer bars welded to the longitudinal steel at the four corners. The bars were of intermediate grade steel with the following average mechanical properties, as determined from tests of several bars of each size.

Tensile yield point	45,800 psi.
Tensile strength	73,700 psi.
Elongation in 8 in.	25 per cent

In addition to the reinforcement as above described, one half of the slabs had reinforcement in the form of overlapping helical coils of either $\frac{3}{16}$-in. or ¼-in. wire, and of such diameter as to fully enclose the main

TABLE 1—SUMMARY OF IMPACT TEST RESULTS

Slab Designation	Thickness	Spiral Wire Dia.	Percentage of Reinforcement (Area)		Weight of Steel Per Sq. Ft. of Slab			Vertical Pendulum Fall at Failure –H–	Relative Impact Resistance	Ratios		Control Cylinder Strength
			Long.	Trans.	Main	Spiral	Total			$\frac{H}{\text{Total Wt.}}$	$\frac{H}{\text{Steel Wt.}}$	
	In.	In.			Lb.	Lb.	Lb.	In.				lb./in.²
1-st	6½	—	4.6	2.0	17.6	—	17.6	14	1.0	0.21	0.79	2840
1-sp4	6½	¼	4.6	2.0	17.6	4.4	22.0	32	2.3	0.45	1.45	2840
1-sp3	6½	3/16	4.6	2.0	17.6	2.5	20.1	22	1.6	0.31	1.09	2760
2-st	4¼	—	4.7	3.1	13.5	—	13.5	14*	1.0	0.30	1.04	3400
2-sp3	4¼	3/16	4.7	3.1	13.5	2.3	15.8	18	1.3	0.37	1.14	3400
3-st.	7½	—	6.7	1.1	23.8	—	23.8	17	1.0	0.21	0.72	2760
3-sp4	7½	¼	6.7	1.1	23.8	5.1	28.9	48	2.8	0.56	1.66	2720
4-st	6	—	12.1	1.4	32.9	—	32.9	18	1.0	0.24	0.55	2800
4-sp4	6	¼	12.1	1.4	32.9	4.1	37.0	38	2.1	0.48	1.03	2800
G4-st	6	—	12.1	1.4	32.9	—	32.9	—	—	—	—	6500
G4-sp4	6	¼	12.1	1.4	32.9	4.1	37.0	—	—	—	—	6500
5-st	6½	—	3.8	2.0	15.7	—	15.7	12	1.0	0.18	0.77	5220
5-sp3	6½	3/16	3.8	2.0	15.7	2.6	18.3	21	1.8	0.31	1.15	5220
5-stx	6½	—	3.8	2.0	15.7	—	15.7	—	—	—	—	4830
5-sp3x	6½	3/16	3.8	2.0	15.7	2.6	18.3	—	—	—	—	4830

*Not determined from permanent deflection.
**Assigning a value of 1.0 to slabs without spirals.

reinforcement and yet permit a concrete cover of about ½ in. outside of the spiral. The number of coils varied from 8 to 11, depending upon their diameter.

The two slabs for the single impact test, designated 5-stx and 5-sp3x, were 6½ in. thick, 30 in. wide, and about 9 ft. long. They were thickened along their short edges in order to concentrate considerable mass beyond the support, the theory being that the inertia of this mass would simulate the effect of continuity of the slab beyond the support. Details of these slabs are also shown in Fig. 1.

All but two of the slabs were constructed of Haydite concrete. One group of slabs contained 6 sacks of cement, and the other 8½ sacks of cement per cu. yd. of concrete, the proportions being 1:1.3:1.0 and 1:0.8:0.6, by dry weight, respectively. The nominal strengths were 5000 psi. for the latter and 3,000 psi. for the former. Sufficient water was added to these mixes to produce a slump of 6 in.

The remaining two slabs were of sand-gravel concrete. These consisted of standard portland cement, well graded concrete sand, and ½-in. maximum size gravel, mixed in the proportions of 1:1.7:1.8, by dry weight, with an initial water-cement ratio of 0.51, also by weight. This corresponds to approximately 8 sacks of cement per cu. yd. and 5¾ gal. of water per sack of cement. The slump for this concrete averaged about 6 in. The term "initial water-cement ratio," as used above, refers to the original water added to the mix. The quantity of water subsequently removed by vacuum processing, to which all of the slabs were subjected, was more or less variable and an unknown quantity.

The slabs were constructed in a vertical position with the forms supported on their long edge and vacuum processed from one face, the vacuum mat serving as part of the formwork. An external type magnetic vibrator was clamped to the form and operated while the form was being filled and for about 10 min. during the 20-min. vacuum processing period. The slabs were given several coats of a moisture seal, commercially known as "Aquastatic," and they were tested at the age of 9 days. Control cylinders (6 in. by 12 in.) vacuum processed in special molds for a period of 10 min., and cured in the same manner as the slabs, developed the 9-day strengths given in Table 1. The processing of the control cylinders in special molds provided by the Vacuum Concrete Corporation is reported to produce a concrete comparable in physical properties to that in a processed slab.

2. Description of apparatus

The specimens were supported in a vertical position against the ends of two massive concrete piers backed by two solid brick piers and a retaining wall supporting an earth embankment. One of the concrete

piers and portions of the brick piers may be observed in Fig. 6(a). Two mine-car rails, fastened vertically to the ends of the pier and spaced 5 ft. apart, served as the bearings for the test specimens. Impact loads were delivered to the slabs by means of a 12-ft. pendulum consisting of a 1,200-lb. pear-shaped cast-iron weight suspended by a chain from a timber framework. The pendulum was pulled back to its starting position by means of a cable and winch, the cable passing over a pulley fastened to the timber frame at a height sufficient to raise the pendulum to a maximum lift of 8 ft. The load was released by severing a wire loop linking the end of the cable to the pendulum bob.

Residual or permanent deflection was measured with a mid-ordinate gage, consisting of a light structural steel angle, approximately spanning the slab, and a 1/1000-in. micrometer dial fastened at its center.

3. Method of testing

The slabs were held in direct contact with their rail supports by a pair of I-beams serving as clamps which were applied to the slab directly opposite the rail bearings. Between the clamps and the outer face of the specimen a short length of rubber hose was inserted so that, when the clamps were drawn up, very little restraint to rotation was introduced at these points. This device also absorbed much of the shock from the rebound of the slab after an impact blow was delivered.

Since the number of test specimens was limited, the impact loads were applied in increasing increments until failure occurred. This method has serious disadvantages for impact testing, as will be pointed out later, but under the circumstances it was the only possible procedure. Except for two slabs, the pendulum was released at 6-in. increments of height starting 6 in. above the center of the slab and increasing up to a height sufficient to cause severe damage to the slab. In the two exceptions, one blow was delivered to the specimens from the pendulum released at 8 ft. In all tests care was taken to prevent multiple blows which would have been caused by the rebound of the pendulum, and only one blow was delivered to the slab for each increment of height, except in tests of the sand-gravel concrete slabs where repeated blows were delivered after the 4-ft. height was reached*. All of the specimens were tested so that the surface from which they were vacuum processed was the one exposed directly to the impact blow.

Residual deflection measurements were obtained on the rear surface at the center of the slab after each blow of the pendulum. During the progress of the tests, notes and photographs were taken describing the

*At the time these tests were made the apparatus was not equipped to deliver a blow from a height greater than 4 ft.

behavior of the specimen, the formation of cracks and the spalling and crushing of the concrete directly exposed to the impact blow. In addition, photographs were frequently taken to supplement the descriptive data.

3—RESULTS OF TESTS

It is difficult to establish a definite criterion for failure in slabs subjected to impact loads. In a reinforced-concrete member subjected to slowly increasing loads, initial failure is generally considered to have occurred if or when its reinforcement has reached the yield point, because the member generally can support but little additional load after this state has been reached. This, however, is not necessarily true when the load is suddenly applied. A relatively small impact load is capable of stressing the reinforcement in such a member above the yield point, and at the same time it seems possible that the static load-carrying capacity may not be appreciably affected. This assumes, of course, that the impact load is applied but once. Furthermore, even if a considerable local yielding of the reinforcement occurs in a structure such as a slab or hull of a ship, it does not necessarily indicate a concomitant failure of the entire structure. It would appear, therefore, that, under these circumstances, failure under impact is governed not so much by the stress in the steel as by the condition of the concrete, the important consideration being that the concrete is reasonably whole and not severely shattered.

In order that comparisons between various slabs tested for impact could be made readily, it was desirable to define failure more specifically by some measured value. A study of the relationship between a given permanent deformation and the general appearance of the Haydite slabs, as determined from photographs and written descriptions obtained during the tests, revealed that, within reasonable limits, a given deformation could be depended upon to indicate the extent of damage suffered. For instance, if a permanent deflection of 0.15 in. were selected as a criterion for failure, the test specimen would generally appear badly cracked on the rear face with cracks ranging in width from 0.01 in. to 0.03 in. and, in the case of slabs without spiral reinforcement, a distinct separation of the concrete from the bars or splitting of the slab along its neutral surface as indicated by a wide longitudinal crack along the top edge of the slab, a type of failure characteristic of all the specimens not containing spirals. The front surface would generally appear badly indented, with some crushing, and in many cases spalling due to flexure.

If a permanent deflection of either 0.10 in. or 0.20 in. were selected as a criterion of failure, the relative impact resistances of the slabs would have been approximately the same as for 0.15-in. permanent deflection. A permanent deflection of 0.15 in. was arbitrarily selected as the criterion for failure of the Haydite slabs.

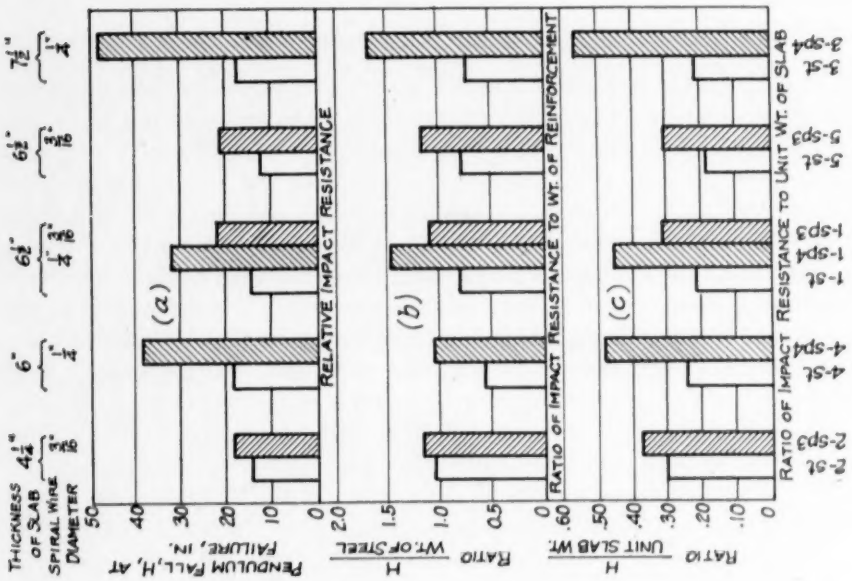

Fig. 3—Permanent deflection curves due to impact

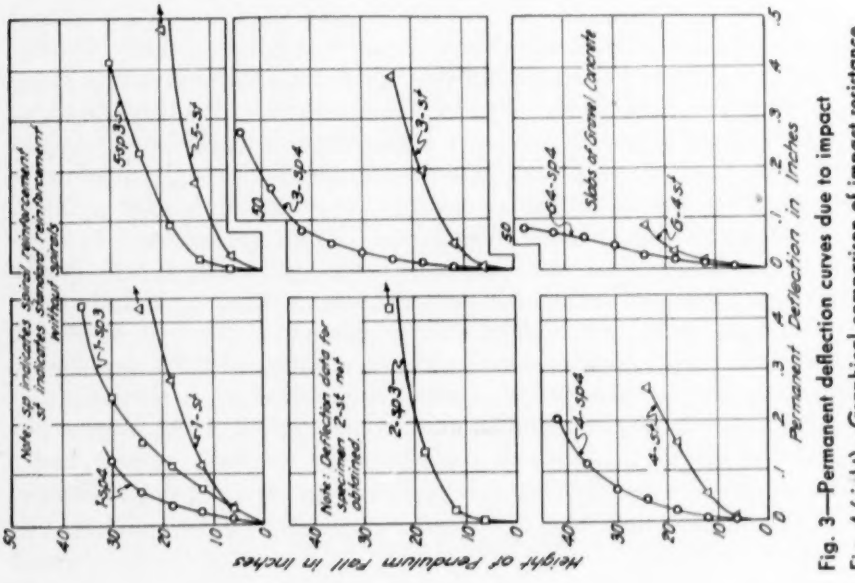

Fig. 4 (right)—Graphical comparison of impact resistance

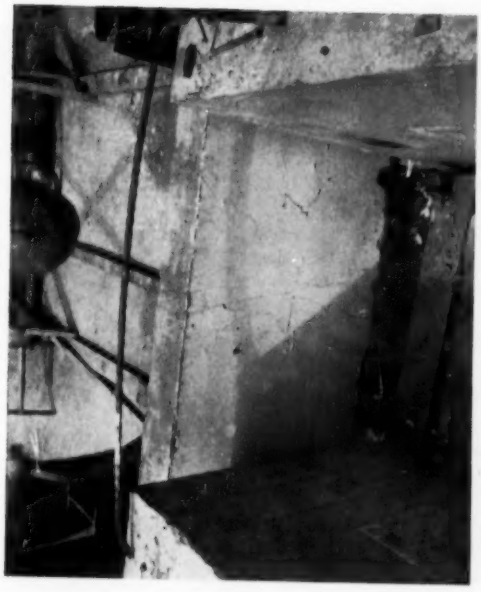

Fig. 5(a) (left)—Specimen 5-sp3x after 96-in. fall of pendulum—front face (with spirals)

Fig. 5(b) (above)—Specimen 5-sp3x after 96-in. fall of pendulum—rear face (with spirals)

Fig. 6(a) left)—Specimen 5-stx after 96-in fall of pendulum —front face (without spirals)

Fig. 6(b) (above)—Specimen 5-stx after 96-in. fall of pendulum—rear face (without spirals)

The curves of Fig. 3, showing the relation between permanent deflection and height of pendulum drop, were used to apply the criterion for failure as above described. It has been assumed that the relationship shown by these curves would be found if the pendulum has been released at heights intermediate beween those actually used in the tests. This method of determining the load which will cause failure, in effect, penalizes those slabs requiring a greater number of load increments to reach a given deflection, since each additional impact blow, regardless of its magnitude, adds to the deflection, and damage as well. In other words, a succession of blows with increasing intensity up to a maximum value is more damaging than a single blow of intensity equal to the maximum value. This fact is effectively demonstrated by the tests which will be discussed later. However, inasmuch as comparisons are made only between slabs that were tested in a similar manner, and as the values defining failure are at best approximate, this procedure appears to be reasonably satisfactory. The values referring to failure are obviously only relative measures and the maximum impact that any slab can withstand before failure, as defined above, may be considerably larger with a single blow.

The effect of spiral reinforcement upon the relative impact resistance of each of the various slabs is indicated in Table 1 and graphically illustrated in Figure 3 and Fig. 4. The slabs containing spiral as well as bar reinforcement exhibited a considerably greater resistance to impact than those with the bar reinforcement only with the exception of the 4¼-in. slabs, which showed only a moderate increase; this was clearly observed from the appearance of the specimens during, as well as after, the tests and is demonstrated by the criterion for failure based on permanent deflection, which indicates an increase of from 1½ to almost 3 times the resistance of companion specimens containing the bar reinforcement.

Fig. 5 shows the extent of damage suffered by the slab with spirals, after a single blow of the pendulum released from an 8-ft. height. The superior performance of the slab containing spirals is well illustrated by comparison with Fig. 6, showing the damage to the slab without spirals, when tested under the same conditions.

The addition of spirals in the 4¼-in. thick slabs was not as effective as in the others, although the comparison may have been somewhat different had ¼-in. spirals been used. There is some evidence to indicate that the 3/16-in. spiral is not as effective as the ¼-in. diameter wire. Differences in the impact strength of specimens 1-sp3 and 1-sp4 appear to bear this out.

Another illustration of the effect of spiral reinforcement is given in Fig. 7 and 8, which show the condition of the sand-gravel concrete slabs,

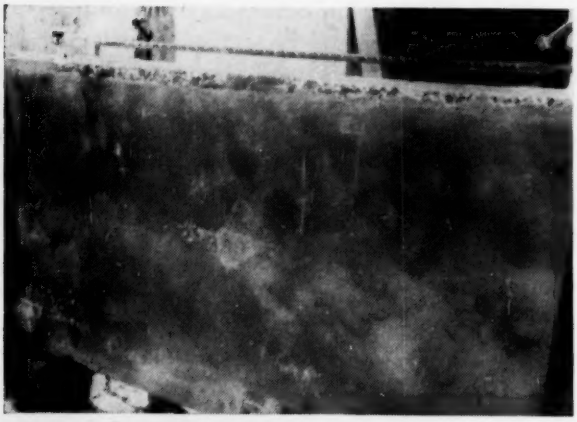

Fig. 7(a)—Specimen G4-sp4 after 48-in. fall of pendulum — front face (gravel-concrete slab with spirals)

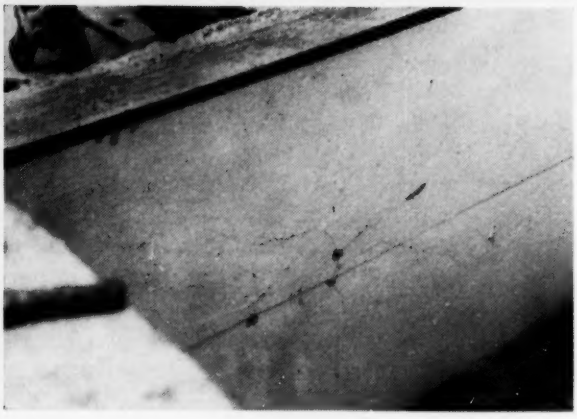

Fig. 7(b)—Specimen G4-sp4 after 48-in. fall of pendulum — rear face (gravel-concrete slab with spirals)

with and without spirals, respectively, after the first blow of the pendulum released from a height of 4 ft. It should be explained that the latter slabs were subjected to a succession of 8 blows of the pendulum of increasing intensity before the photograph was taken, hence the relatively greater damage.

Again referring to Fig. 4(a), it can be seen that specimen 3-sp4, a slab 7½ in. thick and containing spiral reinforcement of ¼-in. diameter wire, exhibited the greatest resistance to impact of the Haydite concrete slabs, which were tested in like manner. Its nearest competitor, specimen 4-sp4, which was 6 in. thick and also reinforced with ¼-in. spirals, was about 80 per cent as resistant to impact. A more significant comparison, perhaps, is the relative efficiencies of the slabs with respect to their weight per sq. ft. of slab, which is an item of importance in ship design, and with respect to the quantity of steel used for reinforcement in each,

Fig. 8(a)—Specimen G4-sp4 after 48-in. fall of pendulum — front face (gravel-concrete slab without spirals)

Fig. 8(b)—Specimen G4-sp4 after 48-in. fall of pendulum — rear face (gravel-concrete slab without spirals)

which also is of considerable importance, particularly at this time. Ratios expressing the relative efficiencies of each design are given in Table 1 and illustrated graphically in Fig. 4(b) and 4(c). They were determined by dividing the height of pendulum fall causing failure, H, by the combined weight of steel and concrete per sq. ft. of slab, or by the weight of reinforcement, including both main steel and spirals, in a square foot of the slab. From this it then follows that the larger the ratio the more efficient the design. An examination of Table 1 and Fig. 4 indicates that the slab excelling in impact resistance also ranked highest with respect to over-all weight, as well as in the quantity of steel used. It is interesting to note that the differences between the various slabs with spiral reinforcement are not nearly so marked in these comparisons, and their relative rank in several instances has changed. For example, specimen 4-sp4 ranking second in relative impact resistance, was relegated to fifth place when judged by its weight of reinforcement. It is, of course,

recognized that these values are based on single tests and that additional tests of each type might alter the comparisons.

A surprising result of the tests was the relatively poor showing of the slabs constructed of higher strength concrete. Although the concrete in specimen 5-sp3 had approximately twice the compressive strength of that contained in the slab most nearly comparable in design, specimen 1-sp3, its impact strength was about the same. The corresponding specimens without spirals also appear to bear out this lack of difference in impact resistance.

Unfortunately, when the gravel concrete slabs were tested, the impact apparatus was not equipped to raise the pendulum beyond a height of 4 ft., which was not sufficient to damage appreciably the slab containing spirals; consequently, the maximum impact load that it was able to withstand was not determined. The greater impact resistance, however, of the slab containing spiral reinforcement has already been pointed out by a previous reference to Fig. 7 and Fig. 8. The specimen shown in Fig. 7 was eventually subjected to eight blows from the 4-ft. height before serious damage was inflicted. A quantitative comparison between the Haydite concrete and gravel concrete slabs, therefore, was not possible because of the different manner in which failure was achieved, and a qualitative comparison may also be open to criticism because of the wide difference in concrete strength although, as previously pointed out, there is some evidence to indicate that differences in compressive strength itself does not materially affect the impact resistance of the slab. However, a description of the appearance of the two slabs containing spirals after a given impact load is included as a matter of interest. The specimens which are compared as those designated 4-sp4 and G4-sp4 constructed of Haydite and gravel, respectively. Both embodied the same reinforcement design, and both contained spirals of ¼-in. wire. The gravel concrete had an average compressive strength of 6500 psi. and the Haydite concrete 2800 psi., as determined from control cylinders. After an impact blow from a 4-ft. height, the gravel-concrete slab had only a slight indentation on the front surface, was generally cracked on the rear face, the cracks being about 0.02 in. wide, and had a permanent deflection of 0.08 in. The Haydite slab suffered considerable spalling on both front and rear faces, a large section of concrete was broken off the lower front edge of the slab, and the slab had a permanent deflection of something over 0.2 in., which, incidentally, is just beyond the point of failure established for Haydite slabs. The front surface condition of the gravel-concrete slab is shown in Fig. 7(a) after a 4-ft. drop of the pendulum and the appearance of the front face of the Haydite slab after a pendulum drop of 3 ft. 6 in. is shown in Fig. 9. The greater resistance

Fig. 9—Specimen 4-sp4 after 42-in. fall of pendulum—front face (with spirals)

to impact of the gravel concrete is undoubtedly due to the harder natural aggregates and the greater stiffness of sand-gravel concrete. It may be questionable whether the weight disadvantage of gravel concrete, particularly where light weight is of major importance, is fully compensated by its better impact resistance. The concrete containing both coarse and fine Haydite aggregate weighed about one-third less than the sand-gravel concrete.

The test results of specimen 5-sp3x clearly demonstrate the effect of a single blow in determining the maximum impact resistance, as compared to successive blows employed in all of the other tests. Referring again to Fig. 5 which shows the damage to this slab resulting from a single 8-ft. fall of the pendulum, it may be judged from its appearance that the 8-ft. fall was perhaps slightly more than required to produce failure of the slab reinforced with spirals. If it is assumed that a single 6-ft. drop of the pendulum was sufficient to cause failure of this slab, and this assumption is probably a conservative one, there is better than a 300 per cent increase in resistance to impact when compared with specimen 5-sp3, which was similarly designed and which failed at a drop of 21 in. after three successive blows of increasing intensity.

In general, a blow of the pendulum released from a height of 12 in. was sufficient to cause the first visible cracks in the slabs. Although cracks were first observed after a 12-in. drop of the pendulum. in several instances they were relatively large and might have developed earlier had there been an intermediate drop between 6 in. and 12 in. On three slabs, 3-sp4, 4-sp4 and 4-st, cracks were not visible until after the 18-in. fall of the pendulum.

In no case was any of the reinforcement ruptured, although the tests were continued until the concrete was severely shattered.

4—SUMMARY OF RESULTS

The important results of the tests may be summarized in the following statements:

1. The use of overlapping spirals, as supplementary reinforcement, increased the relative impact resistance of concrete slabs. The increase varied from about 1½ to almost 3 times that of slabs reinforced with a grid of bars only. The spirals added to the tenacity of the slab in much the same way as shear reinforcement does to concrete beams subjected to static loads.

2. The slab 7½ in. in thickness reinforced with ¼-in. spirals exhibited the greatest resistance to impact and its performance in relation to the quantity of steel used, as well as to its weight per sq. ft., was the best of all of the slabs tested in like manner. It failed after 8 successive drops of a 1,200-lb. pendulum released from heights increasing in multiples of 6 in. up to a maximum height of 48 in.

3. There was some evidence to indicate that the impact resistance of slabs constructed of Haydite concrete is not appreciably affected by a large variation in the strength of the concrete.

4. A slab may be three times as resistant to a single impact load of a given intensity as to a succession of impact loads increasing uniformly in intensity up to the given value.

5—ACKNOWLEDGMENTS

Acknowledgments are due to D. E. Parsons, Chief of Masonry Construction Section, National Bureau of Standards, for many valuable suggestions, and to various members of both the Vacuum Concrete Corporation and the Masonry Construction Section of the Bureau for the assistance in conducting the tests.

Discussion of this paper should reach the ACI Secretary, in triplicate by July 1, for publication in the November Supplement, concluding Proceedings Volume 39.

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7400 SECOND BOULEVARD, DETROIT 2, MICHIGAN (Supplement) November 1943

Discussion of a paper by Ralph W. Kluge:

Impact Resistance of Reinforced Concrete Slabs*

DISCUSSION BY R. D. KARR and AUTHOR

By R. D. KARR†

The author is to be congratulated on the manner of presentation and the factual data contained in this paper.

It occurs to the writer that one or two comments are in order so that certain implied inconsistencies may be remedied.

A statement in the synopsis, and the first paragraph of the introduction, to the effect that the tests were performed for the Maritime Commission is not exactly correct. The inference is very strong that the Commission instigated the proposal to the National Bureau of Standards that such tests be made in order to assist it in certain deductions regarding the effectiveness of a particular type of impact reinforcement.

The Maritime Commission was approached by Vacuum Concrete, Inc., with certain claims and supporting data as to the desirability of permitting the inclusion of spiral reinforcing in the shell slab of concrete ships under construction. The purpose in mind was greatly to increase the resistance to impact of concrete ships which had, in the past, demonstrated weakness in this regard. The Commission was not much impressed with the proposal because of the inconclusiveness of the supporting data submitted. The advocates of this method then proposed that further tests be made by the Bureau of Standards. When the Bureau of Standards requested an opinion from the Commission, it was informed that data obtained from any such tests would be of interest. It was on such a basis that the Bureau undertook the tests described for Vacuum Concrete, Inc.

The Commission was not consulted nor involved in the planning of the tests, nor the design of the slabs; in fact, the Commission would have had no constructive criticism to offer had it been invited to do so.

*ACI JOURNAL, Apr. 1943; *Proceedings* V. 39, p. 397.

†Formerly, Chief, Concrete Construction Section, U. S. Maritime Commission, Washington, D. C., now Naval Architect, War Shipping Administration, San Francisco, Calif.

The novelty of the proposal precluded any intervention by the Commission since it might later be requested to criticise or evaluate the results obtained.

After studying the data and conclusions presented by the author, it appears to the writer that the type of impact reinforcing investigated leaves much to be desired for several reasons. First, continuous circular coils are mechanically ineffective as stirrups functioning as shear reinforcement, which is their real purpose. The tensile stress accumulating in any part of the coils is not terminated or absorbed by any nearby anchorage point or hook. Such anchorage is obtained only by the accumulative effect of bond with the wire itself within the upper half of its circle. Except for the fact that further changes in details would have greatly multiplied the volume of work involved, it seems certain that a series of straight vertical or zigzag stirrups would have proven to be mechanically more effective than spirals. It might have been possible to have made such straight stirrups continuous similar to that of the old type of continuous beam stirrup once in vogue.

The use of a constant pitch of 2 in. to all spirals appears to the writer to be unwise from the standpoint of congestion of reinforcement. This congestion is rather obvious upon inspection of Fig. 2 and even more so when one studies the installation in actual practice as it was attempted on one ship, an installation which was later abandoned.

Due to the scarcity of any comparable data on the problem of impact reinforcing, the value of the paper is unquestioned but, as in all problems of novelty, the initial efforts to solve them only open our eyes to the unfilled gaps in our factual knowledge. When and if further research may be directed toward this problem, the paper under discussion will undoubtedly serve as a helpful stepping stone to a better understanding of the problem.

AUTHOR'S CLOSURE

The author appreciates Mr. Karr's comments concerning the paper and regrets the inference that the Maritime Commission requested the tests. Credit for planning of the tests, however, which generally implies the design of the test specimens, was specifically given to Vacuum Concrete, Inc.

There are two statements of Mr. Karr's to which exception might be taken. It is not entirely correct to state that continuous circular coils are mechanically ineffective as shear reinforcement. The results of the impact tests themselves indicate that coils of reasonably large diameter offer substantial resistance to diagonal tensile forces. It is true they may be less effective than other systems of shear reinforcement, such as those

mentioned in the discussion. Several static load tests of concrete floor slabs, embodying a system of interlocking spirals, conducted at the National Bureau of Standards prior to the impact tests also indicated that the spirals offer considerable resistance to diagonal tension. Unfortunately, the relative merits of this type of shear reinforcement, as compared to conventional stirrups, was not determined.

As for anchorage, it would appear that the spiral lends itself very well to the requirements for effective anchorage. A more likely objection would be the relatively short length over which the spiral is able to resist diagonal tension due to its rapidly changing direction.

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Effect of Time of Haul on Strength and Consistency of Ready-Mixed Concrete*

By GLENN C. COOK†

SYNOPSIS

Strength and slump tests are reported for ready-mixed concrete mixed in stationary mixer and hauled in a standard truck-mixer-agitator for various periods up to 7 hours. One groups of tests was conducted during early spring and another during summer to study effect of temperature. Wash analyses made on concrete samples before starting haul and at end of haul. Effect of retempering by adding water to restore lost slump was studied.

INTRODUCTION

Effect of time of haul on strength and consistency of concrete has been of interest to the ready-mixed concrete industry since its beginning. Information on that factor has been needed as a basis for specifications. Further, a truck is often delayed due to road conditions or breakdown of either the operator's or the contractor's equipment, and it is important to know just how much delay may occur before the concrete is unfit for use.

A number of tests of the effect of length of haul have been made, but none in recent years. A list of some of the more pertinent literature on the subject is appended. That literature records the data on which current specifications and industry practices are based. The question has arisen as to whether these earlier tests, in view of changes in the characteristics of cement and changes in equipment, are still pertinent. A series of tests was planned to provide information toward an answer to that question.

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†American Builders Supply Co., Louisville, Kentucky.

TEST PROGRAM

Tests were made on concrete mixed in a stationary mixer and hauled over the streets of Louisville in a truck-mixer-agitator. The route presented average street and traffic conditions. Samples for test were taken at about 30-minute intervals until the concrete became very stiff. The time of haul ranged from slightly more than 2 hours to more than 7 hours.

Two groups of tests were made. Each group consisted of three batches of concrete, each made on a different day. The first group was made in late March and early April when air temperatures as low as 38 F. were encountered; the second group was carried out in early July during a period when the air temperature was as high as 93 F.

The concrete was made with cement and aggregates from the source of supply normal for the American Builders Supply Co. The cement was a standard portland cement. The aggregates were sand and gravel from the Ohio River, used in a stream-wet condition; the gravel had a maximum size of 1 in. The concrete was proportioned to have approximately 5 sacks of cement per cu. yd. and to have a slump of 4 to 5 in. Each batch consisted of 3 cu. yd. which was mixed for 2 minutes in an 84S Smith tilting mixer before being discharged into the truck-mixer-agitator.

The truck-mixer-agitator was a Jaeger horizontal type with a 135-cu. ft. drum having a capacity of about 2¾ cu. yd. as a mixer and 4 cu. yd. as an agitator. During the hauling the drum was rotated at 4 r.p.m. The truck-mixer drum was in first-class condition. After the truck was loaded and before the hatch was closed, a sample of 1 to 2 cu. ft. was taken. It consisted of a mixture of smaller samples taken through the hatch by means of a post-hole digger from different locations in the batch. Care was exercised to make the combined sample as nearly representative of the batch as feasible. After the first sample was taken the truck was driven over the prescribed route and returned to the point of sampling within about 30 minutes when another sample was taken. This procedure was repeated until the concrete had a reduction in slump from 4 or 5 in. to about 1 in. Samples of aggregates for sieve analyses, unit weights and for moisture and specific gravity determinations were taken as they were being weighed for the batch.

For each sample of concrete slump tests and three 6 x 12-in. cylinders for compression tests at 28 days were made. The cylinders were molded according to ASTM Standards. They were cured immersed in water at room temperature until tested. For the first and the last samples of each batch wash-analyses by the Dunagan method were made. Also, for certain of the samples in Group 1 a portion of the sample was retempered to the original slump and cylinders made for strength tests. Periodic measurements of air temperature, concrete temperature and relative humidity were made.

ACKNOWLEDGMENT

Prof. D. V. Terrell, Head, Department of Civil Engineering, University of Kentucky, and Stanton Walker, Director of Engineering, National Ready Mixed Concrete Association, assisted in planning the test program. Mr. Walker observed the conduct of one day's tests in detail and he also cooperated in the compilation and organization of the data.

Prof. W. B. Wendt, Head, Department of Civil Engineering and W. R. McIntosh, professor of Civil Engineering, University of Louisville, cooperated in the conduct of the test program. One or the other of these men was present at the plant at some time on each day that tests were made and they had a representative present throughout the investigation. All strength tests were made by Professors Wendt and McIntosh.

DISCUSSION OF DATA

The data are summarized in Tables 1 to 7 and Fig. 1 to 4. While the basic data are given in sufficient detail to permit the reader to analyze them as he wishes, the writer's summaries are directed principally to the following factors:

(a) Information on aggregate and proportions (Tables 1 and 2).
(b) Effect of time of haul on strength (Fig. 1 and Tables 3 and 4).
(c) Effect of time of haul on slump (Fig. 2 and Tables 3 and 4).
(d) Influence of temperature on slump (Fig. 3).
(e) Effect of time of haul on grinding (Table 5).
(f) Effect of retempering (Tables 6 and 7 and Fig. 4).

Materials

The standard portland cement was from the same bin for both groups of tests. Tests of eighteen samples by H. C. Nutting Co. showed the following average results: specific surface, 1930; 7-day tensile strength, 400 psi.; 28-day tensile strength 520 psi.; initial set, 3 h. 30 min.; final set, 5 h. 33 m.; sulphuric anhydride, 1.48 per cent; magnesia, 3.58 per cent; loss on ignition 0.61 per cent; insoluble residue, 0.06 per cent. The sieve analyses, specific gravities and unit weights of the sand and gravel are given in Table 1. As already pointed out the samples were taken at the time the concrete was batched. Only one sieve analyses was made for each batch for the sample thus secured. It is to be observed that both the sand and gravel were somewhat coarser for the Group 2 tests than for those of Group 1.

Proportions

The proportions (producing approximately 5 sacks of cement per cu. yd. of concrete) are tabulated in Table 2. It will be observed that the same proportions of dry materials were used for all batches except No. 4. On that day a full batch was mixed in the usual proportions but it was obvi-

TABLE 1—MISCELLANEOUS TESTS OF AGGREGATES

Tests of samples secured as aggregates were being weighed for concrete batches.

Batch No.	Material	Percents Finer Than Each Size							Specific Gravity	Unit Wt. Lb. Per Cu. Ft.
		100	50	30	16	4	½ in.	1 in.		
Group 1										
1	Sand	0.5	7.3	59.0	84.4	100	..	..	2.64	108.0
2	"	0.5	5.9	57.4	76.0	100	..	..	2.60*	110.0
3	"	0.5	5.8	55.0	77.3	98.4	..	..	2.64	109.0
Av.		0.5	6.3	57.1	79.2	99.5			2.63	109.0
1	Gravel	...	...	...	....	0	55	100	2.68	104.7
2	"	...	...	....	....	2	62	99	2.67	107.3
3	"	...	...	....	....	1	58	100	2.68	106.0
Av.						1	58	100	2.68	106.0
Group 2										
4	Sand	0.4	6.0	47.6	70.4	98.6	....	....	2.64	110.0
5	"	0.0	4.6	34.8	71.8	99.6	....	....	2.64	109.0
6	"	0.2	7.1	37.9	69.3	98.8	....	....	2.64	107.5
Av.		0.2	5.9	40.1	70.5	99.0			2.64	108.8
4	Gravel	...	...	....	....	0.2	46.6	86.3	2.68	107.1
5	"	...	...	....	....	0.2	43.8	92.0	2.68	107.7
6	"	...	...	....	....	0.4	45.7	90.2	2.68	105.5
Av.						0.3	45.3	89.5	2.68	106.8

*Doubtful value.

TABLE 2—BATCH WEIGHTS

Weights of aggregate on dry basis; weight of water includes surface moisture of aggregates.

Batch No.	Weight in Lb.				Water Ratio by Volume
	Cement	Sand	Gravel	Water	
Group 1					
1	1410	4176	5940	1004	1.07
2	1410	4176	5940	935	1.00
3	1410	4176	5940	950	1.01
Av.	1410	4176	5940	963	1.03
Group 2					
4	1410	4755	5362	908	0.97
5	1410	4176	5940	904	0.96
6	1410	4176	5940	920	0.98
Av.	1410	4369	5747	911	0.97

ously granular and unworkable and it (3 cu. yd.) was discarded. The reason for the lack of workability appeared to be an excess of the intermediate gravel sizes, although that condition is not clearly revealed by the sieve analyses. At any rate, another batch with a higher proportion of sand was substituted.

The quantity of mixing water shown in Table 2 is reasonably uniform for each group of tests. The amount of added water is known accurately. The accuracy of the calculation of total water depended upon, of course, the accuracy with which the moisture sample was representative of the batch as a whole.

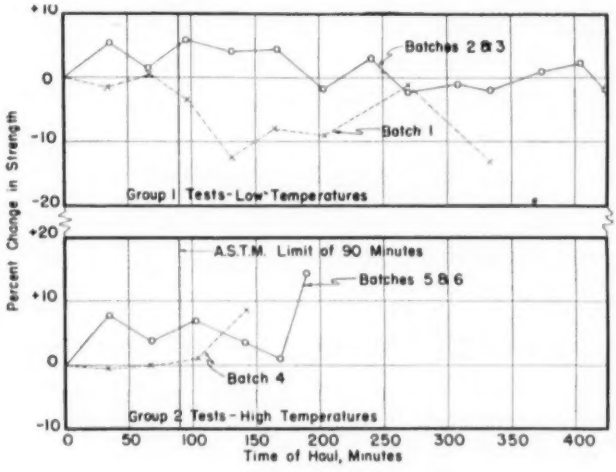

Fig. 1—Effect of time of haul on strength

Strength

The effect of length of haul on strength is summarized in Fig. 1. The detailed data are given in Tables 3 and 4. Where the same time of haul is given for different batches it is accurate to within at least 2.5 per cent or 2.5 minutes. That is to say, for each group of tests, times of haul within 5 minutes, or within 5 per cent of each other, whichever was the greater, were averaged to facilitate tabulation.

Fig. 1 demonstrates clearly that for the conditions of these tests there was no detrimental effect on strength for any normal time of haul. In fact, within the 90-minute limit of the ASTM specification, a limit common in other specifications, substantially all of the strengths after haul were higher than the initial strengths. Further, it is most important to note that for excessive hauls, continued until the concrete had attained an excessively low slump, only isolated reductions of strength in excess of 10 per cent were obtained; there were only two cases (both from the same batch) outside the 10 per cent band and comparisons of these values

TABLE 3—EFFECT OF TIME OF HAUL (GROUP 1—LOW TEMPERATURES)

Slumps are averages of two determinations; strengths are averages of three values, except those marked with asterisk which are averages of two.

Item	Data for Different Times of Haul													
	0	34	66	98	132	167	203	240	271	307	334	375	405	436
Batch No. 1														
Rel. Hum., %	80	63	35	33	28	...	...	...	...	...	...	...	...	...
Air Temp. (F.)	48	50	64	66	69	72	73	...	74	...	75	...	...	...
Conc. Temp. (F.)	58	60	62	64	64	68	72	...	78	...	82	...	...	...
Slump, in.	4.2	4.0	3.2	2.5	2.2	1.8	1.4	...	0.6	...	0.1	...	...	...
Comp. Str., psi.	4300	4240	4320	4140	3760	3970	3900	...	4230	...	3720	...	...	...
Batch No. 2														
Rel. Hum., %	75	67	62	62	51	53	46	42	41	38	...	43	...	36
Air Temp. (F.)	38	40	42	42	44	46	47	48	48	50	...	50	..	53
Conc. Temp. (F.)	54	54	55	56	56	56	58	58	59	60	...	62	...	60
Slump, in.	5.1	4.5	4.1	3.4	4.0	2.9	2.9	2.4	1.7	2.0	...	1.3	...	1.1
Comp. Str., psi.	3930	4100	3940	4280	4120	4270*	3910	4130	3830	3910	...	3980	...	3870
Batch No. 3														
Rel. Hum., %	75	62	62	71	62	61	52	45	45	...	41	...	39	...
Air Temp. (F.)	40	42	44	44	46	47	50	51	52	...	53	...	54	...
Conc. Temp. (F.)	52	52	53	54	54	56	58	58	58	...	60	...	60	...
Slump, in.	5.0	4.7	4.7	4.5	4.6	3.1	3.9	2.5	2.4	...	1.4	...	1.4	...
Comp. Str., psi.	3960	4170	4040	4030	4070	3960	3820	3980	3860	...	3860	...	4030*	...
Average of Batches 2 and 3 (Similar Temperature Conditions)														
Rel. Hum., %	75	64	62	66	56	57	49	44	43	...	41	43	39	36
Air Temp. (F.)	39	41	43	43	45	46	48	50	50	50	53	50	54	53
Conc. Temp. (F.)	53	53	54	55	55	56	58	58	59	60	60	62	60	60
Slump, in.	5.0	4.6	4.4	3.9	4.3	3.0	3.4	2.5	2.0	2.0	1.4	1.3	1.4	1.1
Comp. Str., psi.	3940	4140	3990	4160	4100	4110	3860	4060	3840	3910	3860	3980	4030	3870
Average of Batches 1, 2 and 3														
Rel. Hum., %	77	64	53	55	47	...	...	...	...	...	...	...	...	...
Air Temp. (F.)	42	44	50	51	53	55	57	...	58	...	...	...	...	...
Conc. Temp. (F.)	55	55	57	58	58	60	63	...	65	...	...	...	...	...
Slump, in.	4.8	4.4	4.0	3.5	3.6	2.6	2.7	...	1.6	...	...	...	...	...
Comp. Str., psi.	4060	4170	4100	4150	3980	4060	3880	...	3970	...	...	...	...	...

with companion ones suggest that one of them was more than likely due to errors of sampling, molding or testing.

As pointed out, the cement used was a standard portland cement. It is recognized that general, categorical conclusions cannot be drawn from these data in the absence of tests of other cements and, particularly, high-

TABLE 4—EFFECT OF TIME OF HAUL (GROUP 2—HIGH TEMPERATURES)

Slumps are averages of two determinations; strengths are average of three values.

Item	Data for Different Times of Haul						
	0	34	68	101	136	168	180
Batch No. 4							
Rel. Hum., %	67	62	60	58	50	..	..
Air Temp. (F.)	87	90	90	90	93	..	..
Conc. Temp. (F.)	84	84	94	96	98	..	..
Slump, in.	4.9	3.9	3.2	2.2	0.7	..	..
Comp. Str., psi.	3700	3680	3690	3740	4020	..	..
Batch No. 5							
Rel. Hum., %	53	54	57	57	57	57	..
Air Temp. (F.)	66	68	70	71	71	72	..
Conc. Temp. (F.)	72	76	78	78	80	80	..
Slump, in.	4.2	4.0	2.4	2.0	1.5	1.2	..
Comp. Str., psi.	4180	4550	4220	4560	4440	4440	..
Batch No. 6							
Rel. Hum., %	76	71	62	57	65	..	65
Air Temp. (F.)	70	70	72	72	72	..	72
Conc. Temp. (F.)	70	72	78	80	82	..	84
Slump, in.	5.0	4.5	3.6	2.1	1.6	..	0.8
Comp. Str., psi.	4610	4920	4900	4720	4860	..	5040
Average of Batches 5 and 6 (Similar Temperature conditions)							
Rel. Hum., %	64	62	60	57	61	57	65
Air Temp. (F.)	66	69	71	72	72	72	72
Conc. Temp. (F.)	71	74	78	79	81	80	84
Slump, in.	4.6	4.2	3.0	2.0	1.6	1.2	0.8
Comp. Str., psi.	4400	4740	4560	4690	4550	4440	5040
Average of Batches 4, 5 and 6							
Rel. Hum., %	65	62	59	57	57	..	..
Air Temp. (F.)	74	76	78	78	79	..	..
Conc. Temp. (F.)	75	77	83	85	87	..	..
Slump, in.	4.7	4.1	3.1	2.1	1.3	..	..
Comp. Str., psi.	4160	4380	4270	4340	4370	..	..

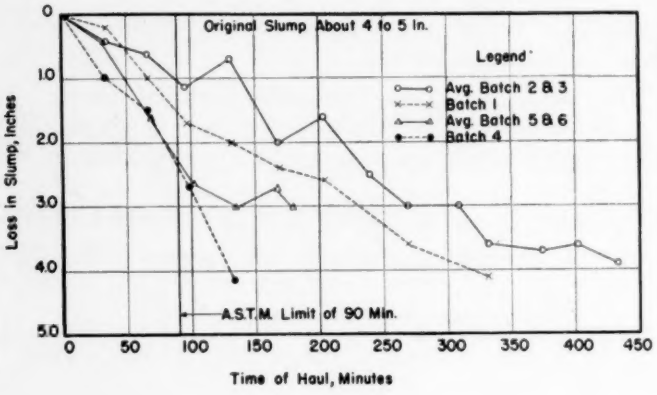

Fig. 2—Effect of time of haul on slump

early-strength cements. However, it must be recognized that these tests confirm results of earlier tests (see appended references) and the comprehensive, but uncorrelated, experience of producers and users of ready-mixed concrete.

Slump

Fig. 2 indicates there was a gradual, but consistent, decrease in slump from the beginning of the haul. See also the detailed data in Tables 3 and 4. The rate of that decrease in slump seemed to be affected principally by the temperature of the concrete, and to some extent by the relative humidity of the air, as will be discussed later. For the early spring tests (Group 1), the slump was reduced about 1 to 1½ in. from the original 4 to 5 in. during a 90-minute haul; for the summer tests (Group 2) the reduction was about 2½ in.

However, it should be emphasized that these reductions in slump were not accompanied by corresponding reductions in workability or placeability. Unfortunately no method for measuring actual workability was (or is) available and the preceding statement must be based on visual observation. As the concrete was hauled it took on a more "fatty" and more plastic appearance and "feel," which did much to compensate, if in fact it did not completely compensate for the loss in slump for hauls of usual length.

It has been suggested that this increase in apparent plasticity might be due to grinding of the aggregates and the cement. Such action may offer a partial explanation. However, for these tests, as will be discussed later, it is believed that grinding was a minor, and perhaps negligible, factor. A principal factor is believed to be the absorption of the mixing water by the cement and the better incorporation of the water with the cement. As already pointed out, the aggregates were "stream-wet" and, therefore, absorbed no water.

Temperature vs. slump

Casual inspection of the data indicate the rate of decrease of slump to be greatly affected by temperature. That suggested the possibility of a relationship such as shown in Fig. 3, where the decreases in slump for each batch and each period of sampling are plotted in relation to a "degree-hours" factor. The "degree-hours" were calculated by multiplying the average temperature of the concrete by the time of haul. For example, if the concrete started out at 58 F, was 60 after 34 minutes,and 62 F. after 66 minutes, the "degree-hours" were calculated as $\frac{58+60+62}{3}$ x $\frac{66}{60}$ = 66, which seems to be a sufficiently accurate short cut to suggest the relationship.

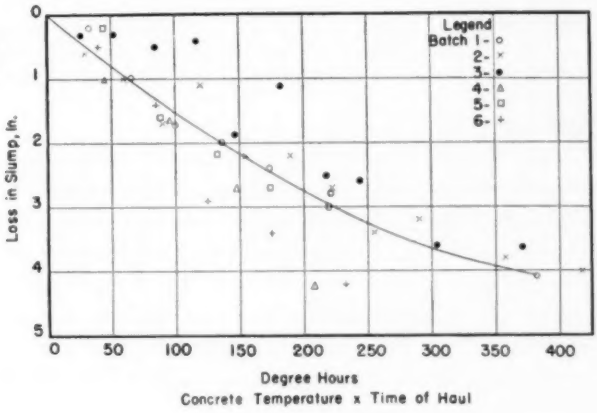

Fig. 3—Effect of temperature on reduction of slump

While the data are somewhat scattering, they arrange themselves in a manner to demonstrate clearly that a relationship exists. While the slope of the curve would undoubtedly vary with materials and equipment, it suggests that individual relationships might be worked out for different conditions of individual operations.

Some effect of humidity may be observed from Tables 3 and 4, the tendency being for the slumps to decrease more rapidly on drier days. However, attempts to correct the "degree-hours" factors for humidity to obtain a better relationship with slump were only moderately successful. It should be pointed out that humidity should be expected to have a greater effect on these tests than on normal operations, since the hatch to the mixer drum was removed every 30 minutes for the purpose of taking samples.

TABLE 5—SUMMARY OF DUNAGAN WASH ANALYSES

Each result is average of three tests.

Test No.	Time of Haul, Min.	Per Cent of Solids, by Weight								
		—No. 100			No. 100 to No. 4			+No. 4		
		Batch*	Begin	End	Batch*	Begin	End	Batch*	Begin	End
1	327	12.2	11.9	13.4	36.2	35.2	33.9	51.6	52.9	52.7
2	436	12.2	11.8	12.8	36.2	35.6	36.5	51.6	52.6	50.7
3	405	12.2	11.9	12.4	36.2	36.5	35.9	51.6	51.6	51.7
Av.		12.2	11.9	12.9	36.2	35.8	35.4	51.6	52.3	51.7
4	135	12.2	11.8	11.6	41.2	39.1	37.5	46.6	49.1	50.9
5	168	12.2	12.3	12.0	36.2	35.4	34.8	51.6	52.3	53.2
6	180	12.2	11.6	12.2	36.2	31.4	29.8	51.6	57.0	58.0
Av.		12.2	11.9	11.9	37.9	35.3	34.0	49.9	52.8	54.0
Grand Average		12.2	11.9	12.4	37.0	35.5	34.7	50.7	52.5	52.8

*Materials as weighed into batch, uncorrected for sieve analyses.

Grinding

An attempt to develop information on the amount of grinding which takes place during agitation was made by means of "Dunagan Analyses" and using the equipment developed by the late Professor Dunagan. For the first and last sample of each batch separations were made, by washing, at the No. 100 and No. 4 sieves. The results are summarized in Table 5. The —No. 100, the No. 100 to No. 4 and the +No. 4 material are expressed as percentages by weight of the total *solid* ingredients. For the record, and for purposes of comparison, the original batch weights are also given expressed in the same manner.

The results of the "Dunagan Analyses" are sufficiently consistent to lead to confidence in their "order-of-magnitude" accuracy. On the other hand, the differences between results before and after hauling are so small, and so erratic within the small range, that they do not permit a quantitative measuring of the grinding. Almost any of the differences encountered might have resulted from normal variations in sampling. The data do suggest, however, that the amount of grinding was small.

Taking the averages of the three batches for each group, and the figures at their face value, it can be reasoned that, in the case of Group 1 tests where the time of haul averaged about 6½ hours, sufficient sand and gravel were ground to increase the —No. 100 portion of the batch about 8½ per cent—an increase of only 2.8 per cent in the "fines" in the sand. Similar reasoning would indicate that no grinding to the —No. 100 sizes took place for the Group 2 tests, where the average length of haul was

TABLE 6—EFFECT OF RETEMPERING ON STRENGTH

Each value average of three tests from same sample.

Batch No.	Test No.	Time of Haul, Min.	Slump, In.			Water-Ratio		Strength, psi.		
			Initial	After Haul	After Retempering	Initial	After Retempering	Initial	After Haul	After Retempering
1	a	198	4.2	1.4	4.4	1.07	1.22	4300	3900	3390
	b	262		0.6	5.0		1.30		4230	3390
	c	327		0.1	4.8		1.33		3720	3110
2	d	307	5.1	2.0	5.8	1.00	1.09	3930	3910	3780
	e	375		1.3	5.4		1.15		3980	3500
	f	436		1.1	5.0		1.16		3870	3620
3	g	277	5.0	2.4	5.2	1.01	1.08	3960	3860	3600
	h	341		1.4	5.6		1.11		3860	3580
	i	405		1.4	4.9		1.14		4030	3620

TABLE 7—STUDY OF RETEMPERED CONCRETE

Rearrangement of data from Table 6.

Test No.	Per Cent Increase in Water-Ratio to Retemper	Per CentReduction in Strength by Retempering	Per Cent Reduction of Retempered Concrete from Initial (No Haul) Strength	Per Cent Reduction in Strength $S = \frac{A}{B^n}$ $(B = 7)$
a	14	13	21	24
b	21	20	21	36
c	24	16	28	40
d	9	3	4	17
e	15	12	11	24
f	16	6	8	26
g	7	7	9	13
h	10	7	10	18
i	13	10	9	22
Av.	14	10	13	24

about 2½ hours. However, at least one inconsistency presents itself to detract from confidence in the results, or, more accurately, from confidence in the applicability of the procedure for the purpose; in the Group 2 tests the amount of +No. 4 material appeared to increase slightly—which could only result from differences in samples.

Retempering

Retempering tests were made only in Group 1. For three samples from each of the three batches, water was added to a portion of the sample and it was remixed by hand. Sufficient water was added to restore the original slump. No retempering was done by adding water to the mixer drum. The results are summarized in Tables 6 and 7 and Fig. 4.

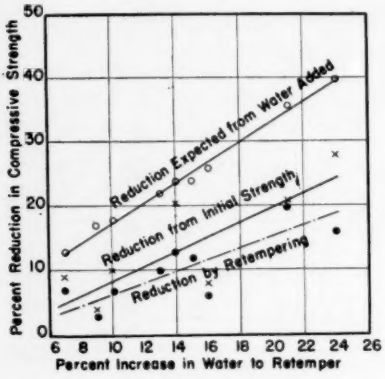

Fig. 4—Reduction of strength by retempering

The reductions in strength due to retempering were considerably less than would be expected under normal circumstances from the amount of water added. The writer has speculated as to the reasons for that discrepancy without arriving at a satisfactory answer; it is too consistent to be an accident, since it occurred for each of the three tests of three different batches—9 tests in all. While the drum was water-tight, and substantially air-tight, the explanation may lie in loss of water by evaporation during sampling.

It will be seen from Table 7 that the average amount of water added was 14 per cent and the average decrease in strength was 10 per cent. That additional water would be expected, from normal water-ratio-strength relations, to reduce the strength about 24 per cent. Further it will be observed, that the average total reduction in strength from the sample with "no haul" to the retempered sample after an average haul of about 5 hours was only 13 per cent. These relationships are brought out in better detail in Fig. 4.

CONCLUSIONS

The following conclusions appear to be justified by these tests for ready-mixed concrete made with a normal portland cement and stream-wet sand and gravel aggregates of the nature of those available in the Ohio River at Louisville, and transported in a standard truck mixer with drum revolving at 4 r.p.m.

1. Times of haul within normal specification limits results in no reduction in strength; generally a slight increase occurs.

2. Excessive durations of haul, continued until the concrete is very stiff, result in strength losses generally less than 10 per cent.

3. Concrete having a slump of 4 to 5 in. immediately after mixing is decreased in slump by a 90-minute haul in an agitator, about 1½ in. when the concrete temperature is about 60 F. and 2½ in. when the concrete temperature is in the neighborhood of 80 F.

4. A good relationship exists between loss in slump and a "degree-hours" factor for the concrete, "degree-hours" being the average temperature of the concrete multiplied by the length of haul in hours.

5. The amount of grinding is negligible for normal lengths of haul.

6. Retempering concrete to restore lost slump appears to reduce the strength only about one half as much as would be predicted from a normal water-ratio-strength relationship.

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Discussion of a paper by Glenn C. Cook

Effect of Time of Haul on Strength and Consistency of Ready Mixed Concrete*

DISCUSSION BY W. C. HANNA, F. B. HORNIBROOK and AUTHOR

By W. C. HANNA†

A study of the paper has convinced me that possibly a mixture with more cement or with different aggregates would have produced results or conclusions that would not agree in every particular with those given. With a rather harsh or lean mixture I would expect the benefits of long mixing to be much more than with a rich mix or one where the fines in the aggregates are rather large. Can Mr. Cook or other members give some additional information?

By F. B. HORNIBROOK‡

This paper gives important additions to the present inadequate information on the effects of hauling concrete in a truck-mixer-agitator for various periods under job conditions. It is only through the correlation of data in several such contributions that the boundaries of safe and unsafe practices can be defined. It is hoped therefore that this study will stimulate similar projects in other localities where prevailing conditions may be different, as for example, where the aggregate used is a different type or of different hardness.

A study of this kind could be expanded almost indefinitely to include such variables as effect of brand and type of cement, effect of grading, surface character and abrasive resistance of aggregates, the effect of wider ranges in temperature, and the effect of length of haul on other important properties of the concrete, such as flexural strength, drying shrinkage and permeability. Since, however, such an extended investigation is beyond the reach of most producers of ready-mixed concrete, it is highly desirable that the maximum amount of pertinent information be

*ACI JOURNAL, Apr. 1943; *Proceedings* V. 39, p. 413.
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gotten from such limited investigations as may be made. Several comments are therefore offered on certain phases of the present study where additional information could have been obtained without excessive additional work, and might have extended the applicability of the results. Perhaps some of this information may be given in a closing discussion but most of the comments may be useful only in the planning of future investigations.

The following items are suggested:

1. A more complete description of the aggregate used would be helpful in correlating results obtained with other materials. A statement of the rock types making up both the sand and the gravel might be given, as well as the result of a Los Angeles abrasion test on the gravel.

2. The sand used was obviously lacking in "fines" and although possibly an excellent sand for use in long hauls where some grinding is bound to occur, yet the sand fails to meet such major specifications as Federal Specification SS-A-281a or ASTM C33-42. All sands in both group 1 and group 2 fail to meet the requirement of the above specification for the amounts passing the No. 50 and the No. 100 sieves, and the sand in batch 1 also has an excess passing the No. 16 sieve. The sands in group 1 also are poorly graded in that approximately 50 per cent of the sand falls in the one fraction passing the No. 30 sieve but retained on the No. 50. The ASTM specification allows acceptance of sands containing a minimum of 5 and of 0 per cent respectively, passing the No. 50 and No. 100 sieves in concrete containing 5 bags or more of cement per cu. yd., provided the approval of the engineer is secured. The sands in group 1, however, would still be disqualified by the "uniform grading" clause of specification C-33-42.

It is not known whether the findings differ significantly from what would have been obtained had a sand grading been used that conformed to recommended gradings. It does pose a question, however, since the use of the sand initially deficient in fines might be expected to have the following effects: (a) change the rate of grinding; (b) improve the grading of the mix by such grinding as did occur. The latter factor would, in the mix used, tend to emphasize the increase in "fatness" after haul and to minimize loss in slump and change in strength due to the favorable effect on placeability.

If the above factors are significant, which is at present only conjectural, the logical solution might be that of specifying sand gradings somewhat deficient in fines where long hauls are to be encountered.

3. The specific weight of the concrete before and after haul should be informative as to yield and other factors. The author in his discussion

under "slump" emphasizes that the reductions in slump were not accompanied by corresponding reductions in placeability since the concrete became more "fatty" and more plastic. Off-hand it appears that three causes might be responsible for the increase in the "fatty" characteristic: (1) increase in fines due to grinding, including increased fineness of the cement; (2) increase in aeration due to agitation, and (3) the obscurely explained but nevertheless commonly experienced phenomenon ascribed by the author. Aeration would be manifested by appreciable decrease in the specific weight, whereas limited grinding might be expected to increase slightly the specific weight.

4. In comparing retempered concrete with original concrete other properties besides strength may be important. If the concrete, for instance, is for use where impermeability or durability is a utilized property, then an increase in voids resulting from an increase in mixing water will produce inferior concrete. If, however, retempering replaces water lost by evaporation only, or if the increase in fines from grinding increases the denseness of a concrete otherwise lacking in fines, then the retempering may not have adverse effect. At any rate consideration might well be given to tests for porosity of the hardened concrete.

5. The data presented in Tables 6 and 7 for the observed reductions in strength after retempering compared to the reductions predictable by the increase in water-cement ratio, naturally stimulates search for a cause of the differences reported. The value of the constant B in the equation $S = \frac{A}{B^x}$ may be one source of the differences. The choice of $B = 7$ indicates, for the nominal 4000-lb. concrete with W/C = 1, that $A = 28000$. The latter figure appears somewhat high even for modern concretes. A calculation of the values of A and B from the expected strengths at various water-cement ratios given in Table 2 of the 1940 Joint Comm. Report on Recommended Practice and Standard Specifications for Concrete and Reinforced Concrete gives A as equal to approximately 18,000 and B as equal to about 6.8. The concrete strengths reported in Table 6 of the present report, however, for a W/C = 1 are much higher than the strengths given in the Joint Comm. Report for the same water ratio. Hence it is difficult to evaluate the constants chosen in the present work. Assuming a lower value of B, however, for instance 5, the average predicted decrease would still amount to 20 per cent as against an average of 10 per cent found.

In searching further for the cause for the differences reported, it is noted that a graph of time of haul against increase in water-cement ratio to retemper gives an essentially straight line for tests a, b, and c, and another essentially straight line of different slope for tests d through i.

It is also noted from Table 3 that batch No. 1 (tests a, b, and c) was subjected to considerably higher temperature than batches 2 and 3. Also a graph of increase in water ratio to retemper (col. 2, Table 7) against difference between found and calculated strengths (column 5 minus column 3, Table 7) gives a roughly linear relationship. These two factors indicate that significant evaporation of mixing water may have occurred, and that part of the retempering water was in fact replacement water rather than increase in water. Thus the longer the haul, the greater the number of times the mixer was opened for sampling (which occurred at about 30 minute intervals) and also the warmer the temperature the greater the rate of evaporation. A superficial calculation, however, based on amount of air in the mixer, assuming saturation of the air and complete change of air every time the mixer was opened for sampling, indicates that less than 2 lbs. of water per batch would have been lost by evaporation.

The paper does not give information on the conditions under which the test cylinders were made, the times required for the slump tests and retempering procedure. It is known, however, that small batches lose moisture rapidly in the open air, hence significant evaporation may have occurred between the fabrication of the "after haul" cylinders and the "after retempering" cylinders. A comparison of the water content in a study of this kind could be made by evaporating to dryness representative samples of the concrete going into each group of cylinders.

In summary, the following suggestions are offered for consideration in future studies in addition to the tests covered in the present investigation:

1. A complete description of the aggregates be given.
2. Aggregate gradings comply with ASTM specifications.
3. A concrete mix be used that is representative of mixes recommended for reinforced concrete.
4. The specific weights of the concrete be determined.
5. The absorption and saturation coefficient of the hardened concrete be determined.
6. A determination be made of the water contents of concrete before and after haul and after retempering.

AUTHOR'S CLOSURE

Mr. Hornibrook has suggested some very pertinent problems that should be considered in any further investigations of a similar nature. The number of variables which present themselves in any study of the behavior of concrete makes it difficult to decide just which should be stressed in a plant study in view of the limited facilities for research

generally available to the ready mixed concrete producer. Our test program was planned with two purposes in mind: we were interested in finding out just what happened to our concrete during different times of haul and under different temperature conditions using the materials that are available here; and we hoped that our work would cause others to make similar studies using their materials. If a sufficient number of independent investigations were made the results would, no doubt, be of considerable value.

Of the additional characteristics suggested by Mr. Hornibrook for determination, it seems likely that volume change due to moisture change may be among the more important. A number of students of concrete have felt that longer mixing results in better assimilation of the water by the cement with accompanying reduction in drying shrinkage, and more thorough hydration of the cement. Some evidence in support of this is offered by the practice of using retempered cement paste to cap cylinders for compression tests; experience has shown that a cap made of freshly mixed paste is more likely to warp and crack than one made of cement paste that has been allowed to stand for some time before use. Other characteristics such as durability, modulus of elasticity, resistance to abrasion, etc., suggest themselves.

The author agrees with Mr. Hornibrook that a more complete description of the aggregates used would be helpful in correlating results obtained with other materials. While no petrographic analyses or abrasion tests were made of the particular lot of gravel used, data pertinent to this point are given in the paper on "Selection of Aggregates for Concrete Pavement Based on Service Records" by Curtis Cantrill and Louis Campbell of the Testing Laboratory of the Kentucky State Highway Department in the 1939 Proceedings of the ASTM. It is not known that this report included results of tests of materials from the identical source that the author used but it is known that it covered materials of a very similar nature.

That paper gives the following mineral composition for Ohio River gravel:

	Per Cent
Granite	34.8
Quartzite	17.9
Sound Chert	12.1
Basalt-Diorite	11.8
Sandstone	11.8
Ferromagnesiums	6.5
Quartz	2.8
Limestone	3.3

Two values of bulk specific gravity are given for Ohio River gravel—one of 2.65 and the other of 2.67. The 3-hour boiling water absorption of

these samples were 0.8 and 0.6 per cent respectively. Two values of the Los Angeles abrasion test are given—25.5 and 20.1 per cent.

Certain routine examinations of gravel from the same source as used in our investigation has been made by the National Sand and Gravel Association Research Foundation at the University of Maryland. The A and B Los Angeles Rattler results were 23.73 and 22.45 per cent respectively on one sample.

These data, as well as general knowledge and observation, indicate the gravel to have good resistance to abrasion.

As pointed out by Mr. Hornibrook the gradation of the sand was not what is generally considered desirable, particularly because of the large concentration of material of the intermediate sizes. The data for the gravel suggest that the sand is hard and resistant to abrasion, it coming from the same source as the gravel. Such grinding as resulted probably had a more beneficial effect on the concrete than would have been the case if more fines had been present. Our tests indicate that relatively little grinding of aggregates took place in spite of the fact that the concentration of intermediate sizes may have encouraged grinding. The author's opinion is that the "fattier" appearance of the mix was mostly due to a better assimilation of the water by the cement and increased fineness of the cement.

It is doubted if aeration due to agitation was a factor. While no unit weight determinations of the fresh concrete were made, each 6 x 12-in. cylinder was weighed before it was broken. The cylinders were molded in cardboard forms, because metal forms in sufficient quantity were not available, and probably some variations in dimension of these forms occurred, which would, no doubt, make small differences in weight of no significance. However, it is believed that these weights may be considered accurate enough to demonstrate that there was no entrainment of air such as might be expected from, for example, a "grinding aid" cement. A brief sketch of some of the cylinder weights indicate that:

(a) For batch 1 the average of three 6 x 12-in. cylinders increased gradually from a weight of 29.72 lb. at "0" haul to 29.98 lb. at 132 minute haul, after which they gradually decreased to an average weight of 29.15 lb. at 327 minute haul.

(b) For batch 2 the average weight of three cylinders started at 29.60 lb., reached 29.95 lb. at 99 minutes and decreased to 29.37 at 424 minutes.

(c) Batch No. 3 showed no significant variations in weight through 200 minutes after which there was a gradual decrease from 29.90 lb. to 29.45 lb. at 405 minutes.

Mr. Hornibrook's discussion of the water required for retempering points to a factor which should be examined further. It is likely that

some of the water required for retempering replaced that lost by evaporation during sampling and handling. If that is the case, the differences in the actual and predicted reductions in strength shown in Table 7 and Fig. 4 can be partially explained. It is true also, as he points out, that the value of B selected in the equation $S = \frac{A}{B^x}$ may have an important bearing on this discrepancy. Perhaps a lower value of B would be more in keeping with modern cements and that also would reduce the discrepancies.

Variables in addition to those suggested by Mr. Hornibrook might well be studied, such as:

(1) Effect of time at various mixing speeds in the truck mixer.

(2) Effect of time at various agitating speeds in a truck mixer or agitator.

(3) Effect of size of batch in relation to volume of mixing drum.

(4) Effect of type of mixing equipment—the high discharge type, the horizontal drum type, the paddle type, etc.

Mr. Hanna suggests the possibility that a mixture with more cement and made with different aggregates would produce results or conclusions that would not agree in every way with what we found in these tests. It is reasonable to believe that a mixture with considerably more or less cement, or one using sand with a large amount of fines, or aggregates with lower resistance to abrasion, might produce different results. However, until results of such tests are available the matter is largely conjectural.

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Oregon Tests on Composite (Timber-Concrete) Beams*

By CONDE B. McCULLOUGH†

Member American Concrete Institute

SYNOPSIS

Tests of timber-concrete beams were prompted by the desire of the Oregon State Highway Department to develope a short span highway bridge intermediate in cost between the *untreated timber* trestle and the reinforced concrete viaduct. The tests are briefly reported here. Important among the conclusions: The ultimate strength of a composite beam (of the type tested), suitably designed, is at least twice that for the same materials and the same dimensions independently employed. The composite bridge design developed is now represented by more than 180 structures with a total length of 20,000 ft.

A most interesting and instructive description of tests on composite (timber-concrete) beams at the University of Illinois was published in this JOURNAL under the authorship of Prof. F. E. Richart and C. B. Williams Jr.‡ Because of the present widespread interest in the composite utilization of these materials, and because certain similar tests, conducted in Oregon, were referred in the Richart-Williams paper, the present writer was requested to present a digest of the Oregon tests.

The test specimens utilized in the Illinois tests might be termed a "slab type" comprising a laminated timber tension segment capped by a concrete compression segment of the same width and connected thereto by means of several different types of shear connections (triangular metal plates, plates and spikes, spikes and daps, lag screws and standard railroad spikes being employed). As distinguished from this type (which obviously embraces a wide field of utility, being competent to carry stress transversely as well as longitudinally) the Oregon test specimens were of the "tee beam" or girder type and the design of the various shear connections was somewhat different.

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†Assistant Chief Engineer, Oregon State Highway Department.
‡ACI JOURNAL, Feb. 1943; *Proceedings* V. 39, p. 253.

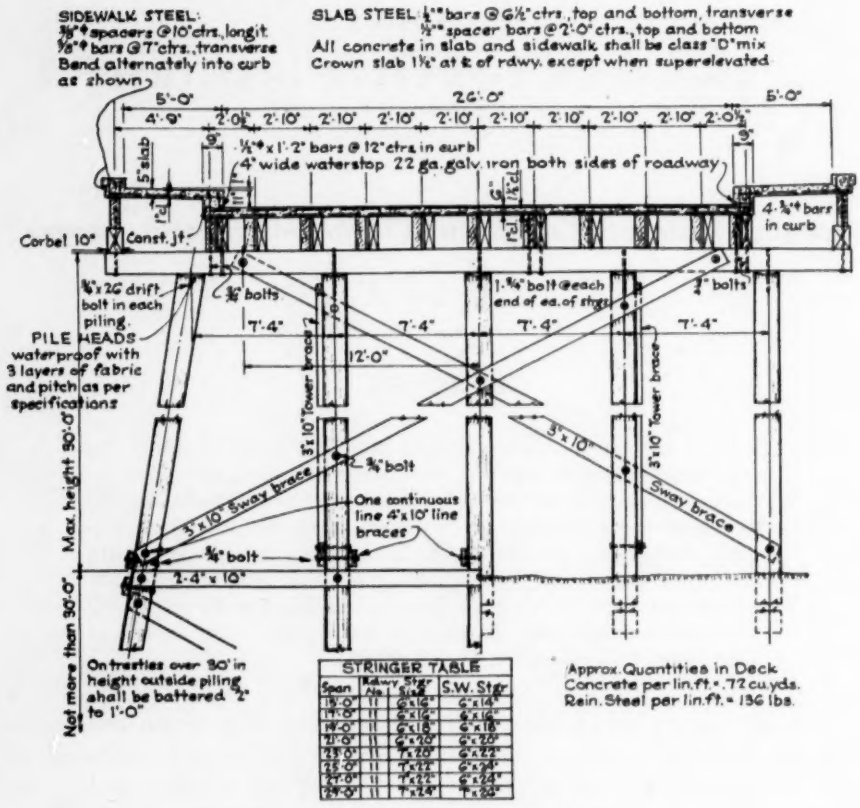


Fig. 1—Half cross-section typical bent

The Oregon tests, in general, were prompted by a desire to evolve or develop a short span highway bridge design in what might be termed an intermediate cost range, the short spans hitherto employed appearing inadequate to meet all needs. At one end of the design group was the *untreated timber* trestle, cheap in first cost, but objectionable from the standpoint of fire hazard, short service life, and high maintenance cost. At the other extreme was the *reinforced concrete viaduct*, advantageous in point of long life, low maintenance expense and architectural excellence, but somewhat higher in first cost than was warranted under some conditions, and in some localities, especially where long approach construction, crossings of sloughs, overflow channels and the like were necessary.

To meet the need a type comprising essentially the composite utilization of a creosoted timber substructure and stringer system with an integrally supported concrete deck (Fig. 1.) was tentatively developed. The advantages apparent for a design of this kind were (1) the overhanging

concrete roadway acts as a waterproof and fireproof covering thus lessening both fire hazard and the tendency toward decay, (2) the monolithically placed deck adds stiffness to the stringer system eliminating in large measure excessive deflections under load with the consequent tendency toward the formation of cracks in the roadway surface, (3) the integration of the two materials renders it possible to space expansion joints at a considerably greater distance than heretofore, thus greatly enhancing the riding properties of the deck, (4) the concrete deck is made to act as a tee beam flange for each of the stringers, thus materially reducing unit stress and therefore adding to the life of the structure since high stresses in timber increase its tendency to check and thus promote disintegration from decay. (5) first costs can be reduced to a point sufficient to render the type economically sound for many locations.

The above advantages were sufficient to render the development of a structural type of this character an attractive *possibility*. However, the entire question hinged upon the degree of dependence which could be placed upon the shear connection between concrete and timber for which reason the researches hereinafter described were inaugurated and carried to conclusion in the laboratories of Oregon State College at Corvallis as a joint research project by the college and State Highway Department.

Briefly, the research comprised the test to destruction of 22 full size composite beams with suitable measurements, for each increment of loading, of deflection, fiber distortion and shear slippage. In addition, certain beams were subjected to repeated or alternated load tests and also to alternated temperatures varying from 15 to 69 F. Two series of beams were tested, series 1 comprising a 6 x 15-in. concrete flange rigidly connected to a 4 x 14-in. timber web while series 2 (Fig. 2) included composite beams having a concrete flange 6 x 24 in. in dimension in combination with a 6 x 16-in. timber web. As will be seen from the illustration the test beams were 16 ft.-0 in. in overall length and 15 ft.-0-in. between centers of supports. The loads were applied at the third points.

Five types of shear connections were employed, as indicated in Fig. 2, method A, with $\frac{3}{8}$ x 8 in. spikes; method B, daps in the timber; method C, a combination of spikes and daps; method D, pipe dowels; and method E, flat steel plates. The size, spacing and general arrangement of the various shear connections for the series 2 beams are shown.

The specific objectives of these tests were: (1) to determine the relative ultimate flexural strength and also the relative deflection of the composite beams in comparison with plain timber beams of equal dimension and under equal load; (2) to compare the various methods utilized for the

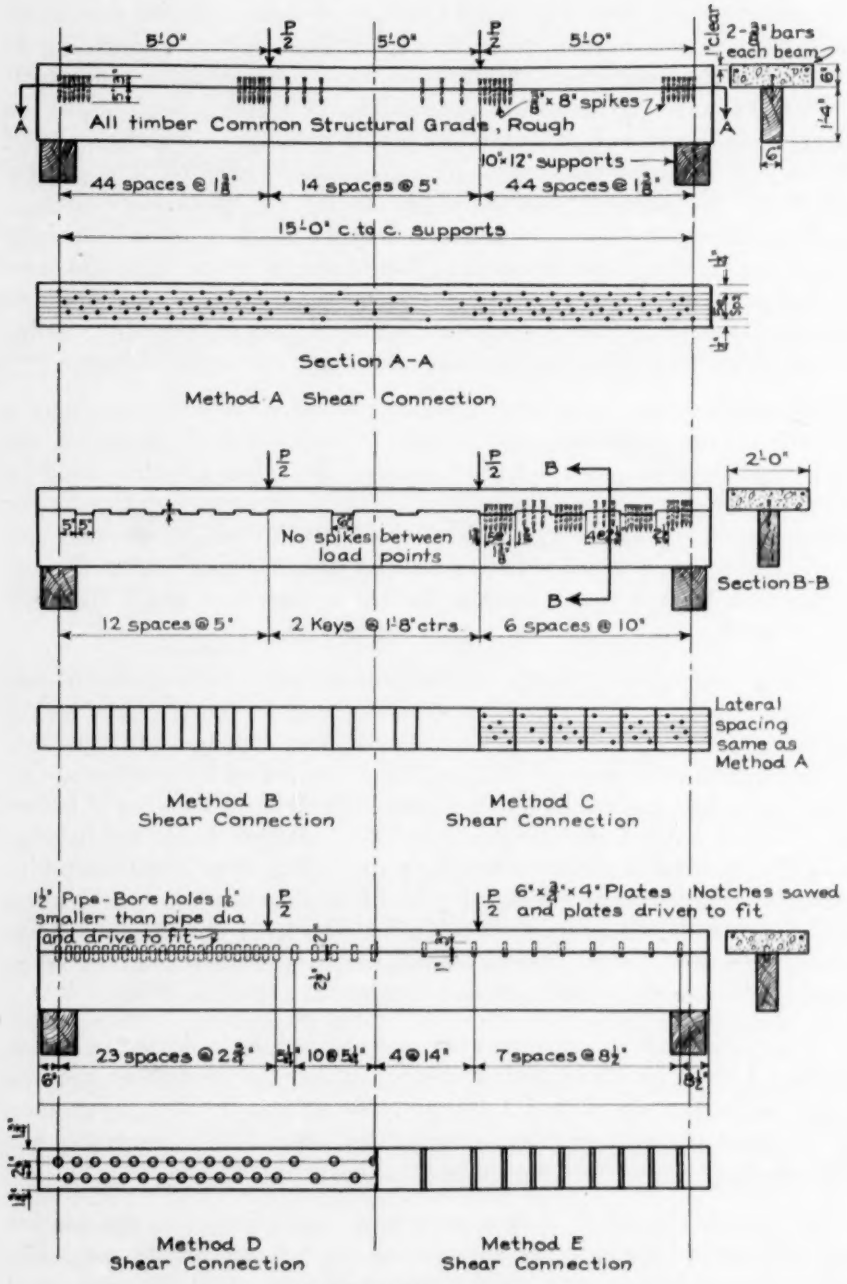


Fig. 2—Test beams, series 2

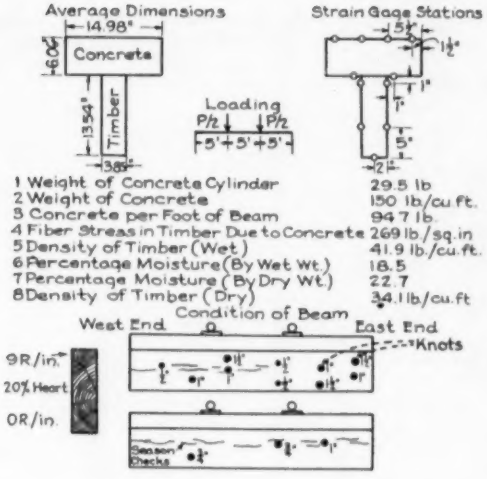


Fig. 3—Beam 1, series 1, failed by horizontal shear at 56,000 lb. load

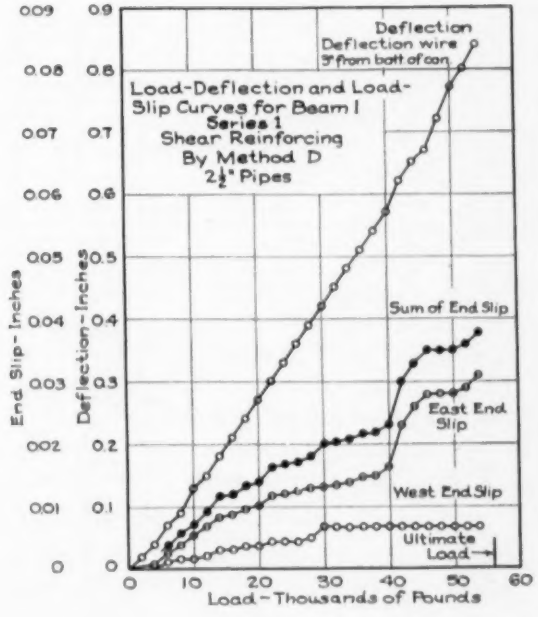


Fig. 4

shear connection and from this comparison to derive recommendations regarding design details; (3) to investigate the effect of repeated or alternated loads; (4) to invesitgate the effect of thermal changes; (5) to study in general the action of the composite beams under load, particularly the location of the neutral axis or axes under various load increments, the character and extent of shear distortion both elastic and plastic and

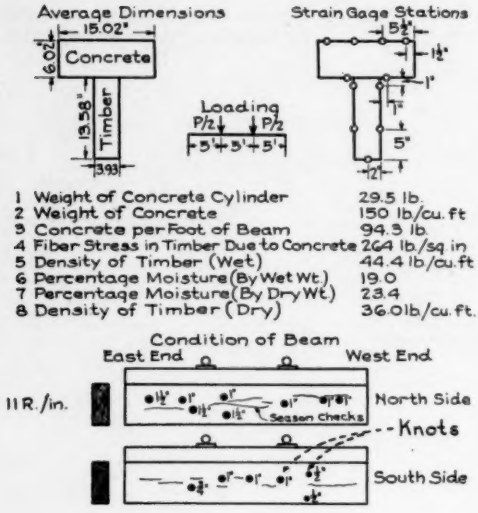


Fig. 3A—Beam 2, Series 1, failed by tension at 64,000 lb. load

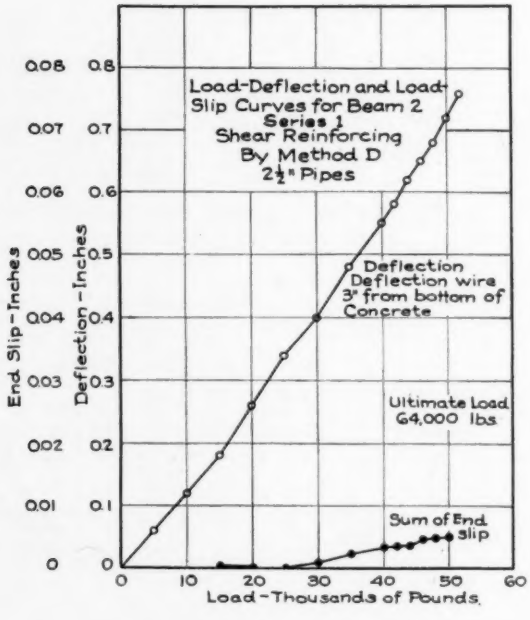
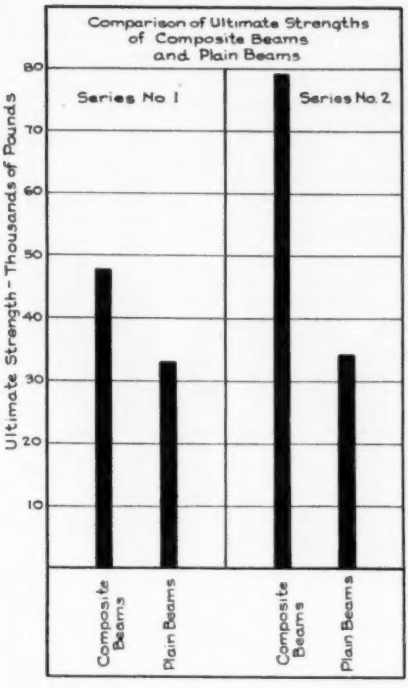


Fig. 4A

(6) from all of these data to evolve a *theory of design* for the composite structure.

Fig. 3 indicates the average dimensions, location of strain gauge stations, and other general data pertinent to beam No. 1, while Fig. 4 indicates the load-deflection and load-end slip relationships for this same beam. Fig. 3A and 4A show similar data for test beam No. 2.



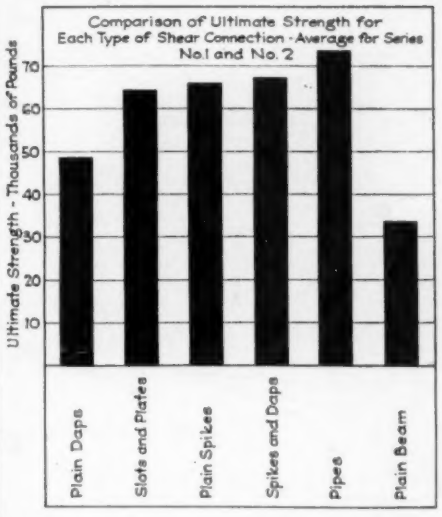


Fig. 5 (left)

Fig. 6 (above)

Fig. 5 is a graphic comparison of the ultimate strength of the composite with that of plain beams having corresponding stem dimensions, but without any upper flange. Fig. 6 indicates a similar comparison grouped in accordance with the type of shear connection employed. Fig. 7 is a graph for test beam No. 5 showing the shear slippage along the junction plane (between timber and concrete) and the resultant development of two separate neutral axes. Similar data for test beam No. 16 are indicated in Fig. 8. This shear slippage and consequent formation of two distinct neutral axes is a phenomenon which closely parallels that described by Richart and Williams in the Illinois tests.

The deviation of the two separate neutral axes from the theoretically computed position varied with the type of shear connection employed as indicated in Fig. 9, it being observed that the spikes and daps, slots and plates, and pipe dowel connections were superior in this regard to either of the other two devices employed. Fig. 10 and 11 indicate for test beams 1 and 2 respectively a comparison of the measured stresses (both tensile and compressive) with those calculated by means of the ordinary tee beam formulas on the hypothesis of no slippage at the junction plane. The graphs marked "theoretical" were calculated on the basis of the *actual* moduli of elasticity of the materials for the test beam in question

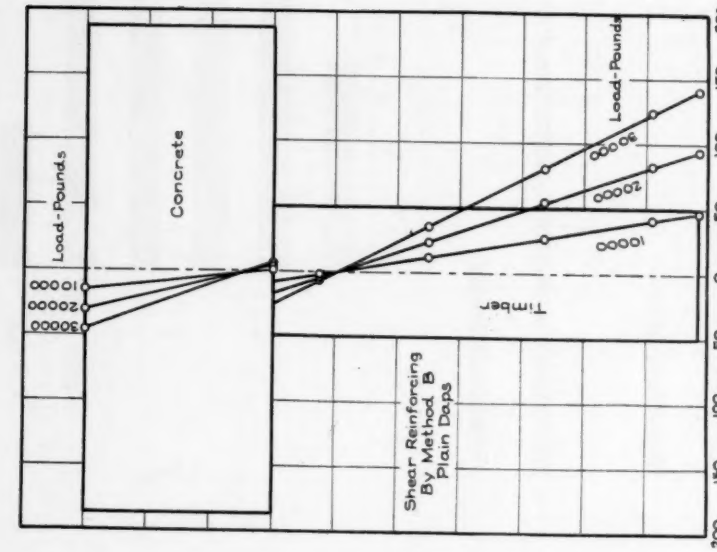


Fig. 8—Location of neutral axis for beam 16, series 1.

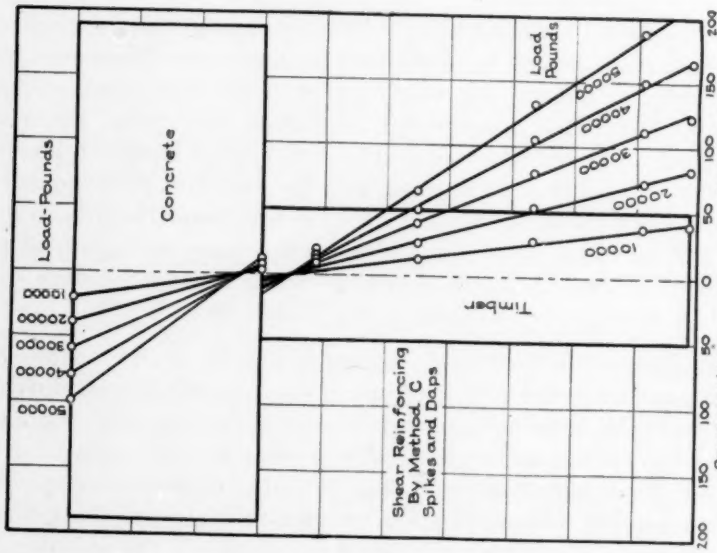


Fig. 7—Location of neutral axis, beam 5, series 1.

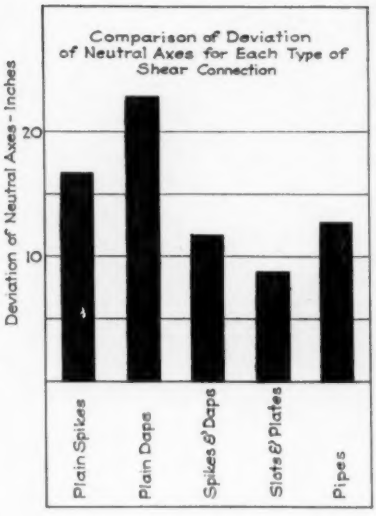


Fig. 9—This comparison is for an average of beams of both series

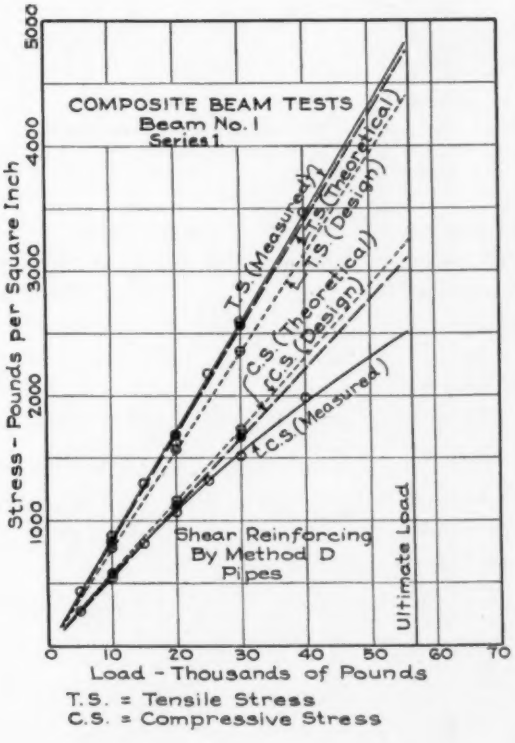
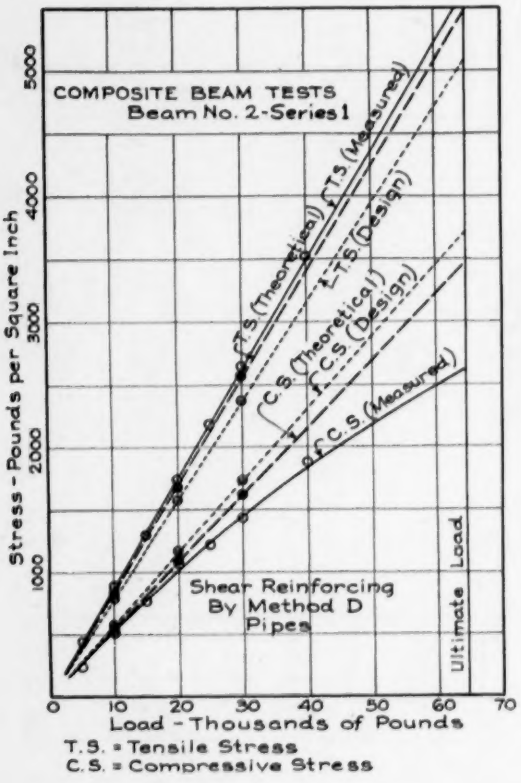


Fig. 10

Fig. 11



so that the variation between these curves and the graphs labelled "measured" represents the discrepancy introduced by shear slippage at the connection plus such small experimental errors as may exist. The graphs marked "design" have been produced in exactly the same manner as the "theoretical" stress curves except that in lieu of the *actual* moduli of elasticity an assumed modulus of 3,000,000 psi. for concrete and 1,500,000 psi. for timber has been employed.

The foregoing description has been, of necessity, somewhat sketchy. Complete data regarding these load tests were published by the Oregon State Highway Commission.* As a result of these tests the following conclusions were stated:

1. The ultimate strength of a composite beam of this type suitably connected at the junction plane and suitably designed is at least twice

*Technical Bulletin No. 1, "Loading Tests on a New Composite-type Short-span Highway Bridge Combining Concrete and Timber in Flexure" was originally published by the Oregon State Highway Department in 1933. The first edition exhausted, the bulletin was revised and reprinted in 1941 and is still available for distribution. It contains 160 pages and 174 illustrations. The four sections comprise (1) introduction; (2) complete description of the full-size composite beam tests; (3) general data regarding this type of bridge structure, and (4) a section pertinent to the economic analysis of the composite type.

Fig. 12 and 13—Composite structures in Oregon
In upper view substructure consists of creosoted timber piling and caps; superstructure a system of creosoted timber stringers integrally connected to a reinforced concrete deck; handrail a concrete rail cap and intermediate posts with ornamental timber spindles. In lower view substructure consists of concrete bents

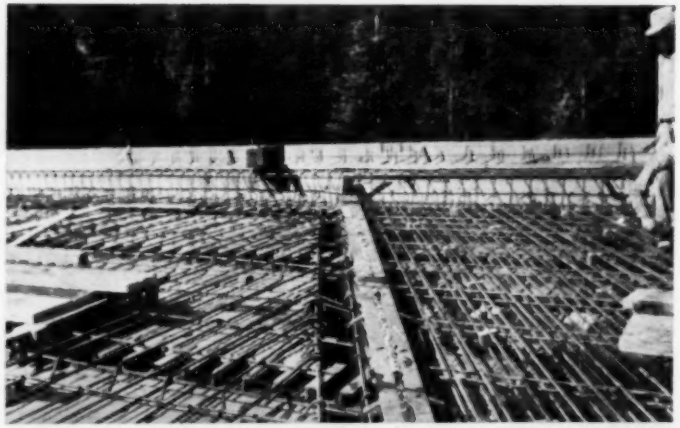


Fig. 14 and 15—Before placing concrete roadway deck
The shear connection in the top view is a system of pipe dowels and in the bottom picture, a system of spikes and daps

that for the same materials and the same dimensions independently employed.

2. The deflection of such a composite beam under a given loading will be not more than 25 per cent of the corresponding deflection for the same sizes and materials used independently.

3. As regards the type of shear connection, the metal pipes and the combined spikes and daps produced the best results all things considered, with the combined spike and dap method presenting the further advantage of greater economy in first cost.

4. Repeated or alternated loads did not appear to have any detrimental effect on this type of construction.

5. Variations in temperature produced stresses of sufficient magnitude to warrant a definite provision for the same in the design. This feature, however, can be adequately taken care of through a proper design of the shear connection. The secondary bending resulting from thermal effects does not appear to induce stress of sufficient magnitude to cause concern; however, the development of tension due to restraint in the lower portions of the concrete slab points to the advisability of an adequate percentage of longitudinal temperature reinforcement.

6. The ordinary transformed section formulae yield results which, within working limits, are sufficiently accurate for all practical purposes.

Based upon the results of the tests a composite design was developed by the Oregon Highway Department and to date more than 180 structures, having a total length in excess of 20,000 feet, have been constructed in accordance therewith. Photographic views and construction details of this type are indicated in Fig. 12 to 15. Through a proper combination of timber and concrete it has been possible to produce a bridge type susceptible of satisfactory architectural treatment and one which, in comparison with the reinforced concrete viaduct, has shown economy in many instances especially in structures in which the distance between grade and ground line did not exceed 25 to 30 ft. or for crossings of sloughs, swamps, overflow channels or for many of those cases where foundation conditions were such as would have rendered deep footings or piling necessary had concrete or steel viaduct construction been employed.

Discussion of this paper should reach the ACI Secretary, in triplicate by July 1, for publication in the November Supplement, concluding Proceedings Volume 39.

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Fatigue Tests of Light Weight Aggregate Concrete Beams*

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Member American Concrete Institute

SYNOPSIS

This paper presents the results of a limited number of reversed and repeated loading tests of unreinforced concrete beams made with Haydite and Gravelite light weight aggregates. True endurance limits were not determined but the results give an indication of the endurance strength for a limited number of repetitions. Light weight aggregates have been used in bridge floor slabs and are now being used in concrete barges where cyclic loadings prevail. Hence, it would seem timely to publish available data on the fatigue resistance of this type of concrete.

A number of fatigue tests have been made to determine the resistance of plain and reinforced concrete beams to repeated and reversed loading conditions.‡ In general, the results have indicated that the endurance limit of plain concrete is 50 to 55 per cent of the static modulus of rupture. The endurance limit of reinforced beams is governed by the fatigue resistance of the reinforcing steel.

Since, in recent years, light weight aggregate concrete has been used in structures subjected to repeated and reversed loading conditions, it seemed desirable that some knowledge of its fatigue resistance should be obtained.

Scope of tests

A series of tests have been made in the Materials Testing Laboratory at Stanford University under the writer's direction to obtain an indication of the endurance limit of concrete made with Haydite and Gravelite. Both are burnt shale aggregates which have been used in important structures.** The program included reversed and repeated loading tests of

*Received by the Institute Jan. 8, 1943.
†Associate Professor of Civil Engineering, Stanford University.
‡Moore & Kommers, "The Fatigue of Metals", McGraw-Hill Book Co., 1927, Chap. XI.
**See "Light Weight Concrete Pavement on the San Francisco-Oakland Bay Bridge" by Glenn B. Woodruff, ACI JOURNAL, Jan.-Feb. 1938; *Proceedings* V. 34, p. 225, for description of Gravelite.

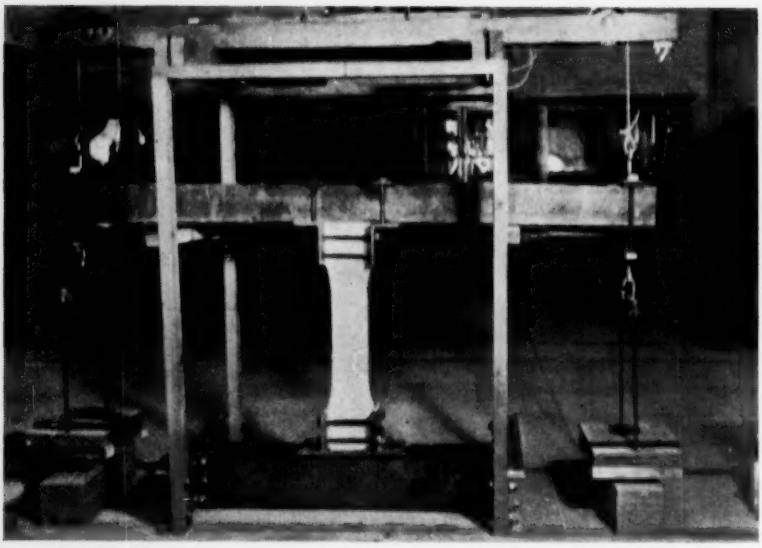

Fig. 1—Fatigue testing apparatus

Haydite concrete beams and repeated loading tests of Gravelite concrete beams.

TESTING APPARATUS AND PROCEDURE

The test apparatus (Fig. 1) was built to handle only one specimen at a time. The lower end of the specimen was clamped to a steel base and a timber loading beam was in turn clamped to the top of the specimen as shown in the photograph. Loading pans at each end of the cross beam were raised and lowered by means of cables which passed over sheaves and were attached to a crank arm on a motor driven speed reducer. With equal loads in both pans, the stress was completely reversed each revolution. One loading pan was removed for the repeated loading tests so that the stress varied from zero to a given tension. By careful adjustment of the cables, the impact was reduced to about 5 per cent. The rate of loading was 15 cycles per minute. An automatic trip stopped the motor when the beam failed.

The apparatus shown was used for all tests of Haydite beams. The Gravelite beams were tested in a similar machine in which the load was applied to the crossbeam by means of a helical spring. The speed was 115 cycles per minute, and the impact was between 10 and 20 per cent. This setup was found to be unsatisfactory in several respects and the apparatus was rebuilt for the Haydite beam tests.

The specimens were 32½ in. long and approximately 4 in. wide and 5⅛ in. deep at the reduced section. Specimens contained no reinforcement. The beams were made in pairs from a single batch of concrete. They were removed from the form 24 hours after pouring and placed in a moist room until ready for test. Control cylinders from each batch were cured in a similar manner.

The Gravelite specimens were all tested at the age of 28 days. When the apparatus was rebuilt and the speed decreased for the Haydite series, it seemed desirable to increase the curing period. Also, since some of the tests would require several weeks, it was somewhat impracticable to test all beams at identical ages. Hence, age of specimens at start of test ranged from 28 to 84 days, with an average of 55 days. The alternating load was always taken as a percentage of the static ultimate load of a companion beam of the same age.

Pertinent data concerned with the concrete mixes are given in Table 1. The Haydite repeated loading test series, H-2, was completed before the H-1 series. No fine natural sand was used in the H-2 series and some

TABLE 1

	Test Data		
Test Series No.	H-1	H-2	G-1
Type of Loading	Reversed	Repeated	Repeated
Light Weight Aggregate	Haydite	Haydite	Gravelite
Mix Proportions (by weight)			
Cement (normal Portland)	1	1	1
Fine Natural Sand	0.75	0	.45
L. W. aggregate dry (1-in. max.)	2.38	1.9	2.1
W/C ratio (gal./sack)	5	6	4.9
Slump (in.)	2–3	6–7	1–2
Wt. of concrete dry (Lb. per cu. ft.)	97	97	95
Age of specimen at start of test (days)	28–83	30–84	28
Compressive strength control cylinders (psi.)			
Range	3210–4410	2750–3590	2700–3350
Average	4030	3020	3110
Modulus of rupture of control beams (psi.)			
Range	365–508	290–523	400–580
Average	440	390	556

difficulty with the mix was encountered. A new shipment of aggregate was received for the H-1 series and it was decided to revise the mix as shown in the table, to eliminate bleeding, and improve workability.

The water-cement ratio given in the table does not include moisture absorbed by the aggregates. Absorption, based on a 30 minute period, amounted to 10 per cent for fine, and 5 per cent for the medium and coarse Haydite. For Gravelite the corresponding values were 12 per cent and

9 per cent, respectively. All aggregates were pre-wet for the 30 minute period.

The Haydite concrete was machine mixed from 2 to 4 minutes in a 2 cubic foot mixer. The Gravelite was hand mixed because no mixer was available at the time.

The testing procedure was as follows:

A pair of beams with their control cylinders were removed from the moist room and weighed. The cylinders were usually tested at once although for the longer beam tests, one was tested at the time the beam failed. These indicated little change of strength during the test period. One beam was placed in the apparatus and loaded to failure by dead loading one pan. The other beam was placed in the apparatus and the pans were dead loaded to a predetermined percentage of the load required to rupture the first beam. The repeated or reversed loading tests was then started and continued until the beam failed. Observations were made from time to time for cracks. Failure was taken as the number of cycles required to rupture the beam. In a number of cases, Berry strain gages were attached to the beam and readings were taken at various times.

Check breaks were made on many of the specimens after failure by loading the longer portion as a simple beam with a concentrated load near the center of the span.

RESULTS AND DISCUSSION

The S-N curves for the three series of tests are shown in Fig. 2. Each plotted point represents one test.

An inspection of the figure indicates that the true endurance limits were not found since none of the curves definitely flattens out. It appears that for less than one million cycles, the Haydite beams will not fail if the stress is below 40 per cent and 50 per cent of the static modulus of rupture for reversed and repeated loading, respectively. The corresponding value for Gravelite under repeated loading appears to be about 40 per cent for less than 100,000 repetitions.* However the plotted values have not been corrected for impact. If this were done the Gravelite beams would compare more favorably with the Haydite repeated load series.

Considerable difficulty was encountered in obtaining consistent results, particularly in the H-2 series. For example, four tests were made at the 60 per cent loading and the number of cycles at failure varied from 4400 to 184,000. The lower range specimens were about 15 per cent lower in compressive strength and in static modulus of rupture. An examination

*Tests of Gravelite beams were discontinued shortly after it ceased to be manufactured.

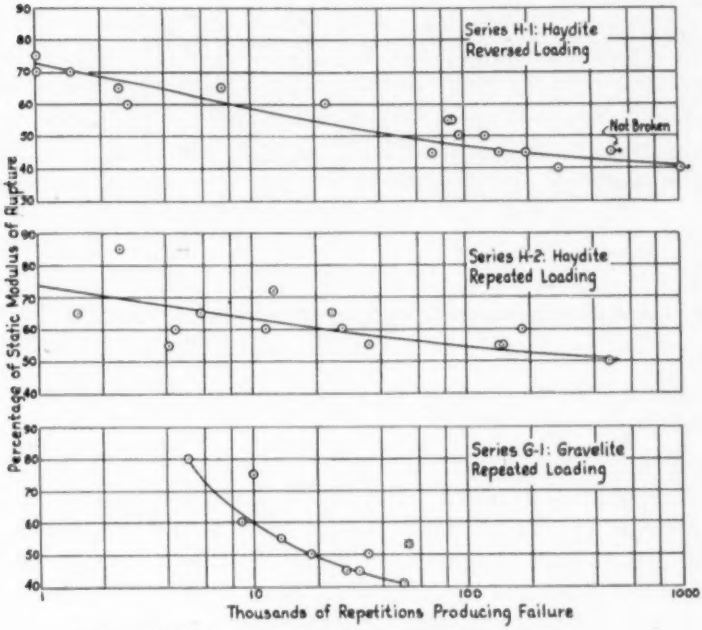

Fig. 2—S-N diagrams for light weight concrete beams

of beam failures revealed that the crack frequently occurred at a point where a weakened piece of aggregate was located close to the tension surface. Some pieces of aggregate were softer than others and some were weakened by cleavage planes. Such points of weakness are particularly significant in the case of alternating loads since, once a crack starts, the stress concentration at the root of the fissure is very high.

Rough corners and surface holes resulting from air and water becoming trapped in the fresh concrete also acted as "stress raisers." In the more prolonged tests under the lighter loads, cracks developed at many of these irregularities and a few gradually extended laterally to the corners and then into the beam. However, the earlier cracks did not always progress until failure occurred because a new crack opened, sometimes well toward the end of the test, and progressed more rapidly. It seems reasonable to believe that, in such cases, a piece of aggregate near the surface failed and developed a more critical stress condition. When the beams were more heavily loaded a crack usually started at a stress raiser and progressed to failure since the stress at the root of the crack was high enough to break through any pieces of aggregate that may have obstructed its progress.

The H-2 series concrete was harsh and the beam surfaces contained many irregularities. Undoubtedly this contributed to the inconsistency

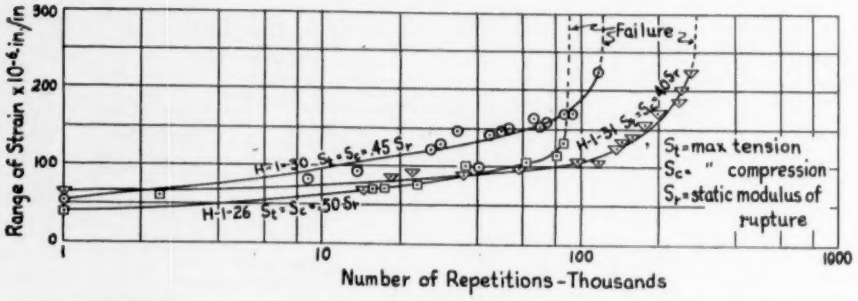

Fig. 3—Curves showing increase in strain for reversed loading tests of Haydite beams, Series H-1

of the results. There was less spread in the H-1 series where the concrete was more workable.

It was further noted that there was no consistent relationship between the static modulus of rupture and the ultimate compressive strength of control cylinders. There was a general trend toward an increase in the former for the higher compressive strength, but there were enough exceptions to give a "shot gun" pattern when one was plotted against the other. Check breaks using a concentrated static load on a portion of the failed specimen usually gave a higher modulus of rupture than the pure bending test. This seems consistent when one considers the bending moment distribution in each case.

The above discussion emphasizes that, while the beams were made in pairs from the same batch, the tensile properties may have been somewhat different. For example, the static modulus of rupture of the control beam might be 400 psi. If the alternating load was to be 60 per cent of the ultimate static load, the tensile stress would be 240 psi. However, because of a particularly weak point, the static modulus of rupture of the latter beam may actually have been 15 per cent lower or 340 psi. so that the specimen was loaded to about 70 per cent rather than 60 per cent of ultimate. In view of this possibility, it is not surprising that the points are somewhat scattered in Fig. 2.

Strain readings were taken on a number of reversed loading specimens with a pair of Berry strain gages. Results of three such observations are shown in Fig. 3. The range of strain refers to the total change in one gage when the stress changed from tension to compression. It will be noted that the larger the load, the more rapidly the curve increases in slope just before complete failure.

SUMMARY

1. Reversed loading tests of unreinforced Haydite concrete beams indicated an endurance strength for one million repetitions of about 40 per

cent of static modulus of rupture for the particular Haydite mix used. The investigation did not determine whether this value would hold for Haydite manufactured at another plant having a different source of raw material. Also, only one particular mix was used throughout this series. The endurance limit may vary with the cement content and the water-cement ratio although the writer is of the opinion that the flexural strength of such beams is governed by the aggregate to a large extent.

2. Repeated loading tests of unreinforced Haydite concrete beams indicated an endurance strength for one million repetitions, about 50 per cent. The tests in this series did not extend beyond approximately a half million cycles. The above remarks regarding the Haydite source, etc., apply to this series also.

3. Repeated loading tests of unreinforced Gravelite beams indicated an endurance strength for 60,000 repetitions, about 40 per cent. The speed of loading, 115 cycles per minute and a higher impact factor probably invalidates any but a rough comparison with the Haydite beams. The two should compare more favorably under the same test conditions.

Discussion of this paper should reach the ACI Secretary, in triplicate by July 1, for publication in the November Supplement, concluding Proceedings Volume 39.

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Vol. 14 No. 5 7400 SECOND BOULEVARD, DETROIT, MICHIGAN April 1943

Job Problems and Practice

"JPP" is a means toward realizing more fully than in a limited number of longer JOURNAL contributions, the mutuality of ACI Membership effort to do a better concrete job. In JPP many Members may participate in few pages. So, if you have a question, ask it. If an answer is of likely general interest, it will be briefed here (with authorship credit unless the contributor prefers not). But don't wait for a question. If you know of a concrete problem solved—in field, laboratory, factory, or office—or if you are moved to constructive comment or criticism, obey the impulse; jot it down for JPP. Remember these pages are for informal and sometimes tentative fragments—not the "copper-riveted" conclusiveness of formal treatises. "Answers" to questions do not carry ACI authority; they represent the efforts of Members to add their bits to the sum of ACI Member knowledge of concrete "know-how."

Repair of Leaky Tunnel Lining (39-128)

Q—We are having some difficulty from the surface and sub-surface water which is leaking through the reinforced concrete lining of a switch yards tunnel. This water filters through a limestone structure which will naturally raise the lime content of the water. Will this additional lime in the water have a tendency eventually to waterproof the lining of the tunnel?

Data in reference to the job are: Design strength, 3500 psi., water-cement ratio, 0.52; cement factor, 1.35 bbl.; no admixture for waterproofing; cylinder break at 7 days, 2430 psi., at 28 days, 3820 and 4240 psi.

A—It is not clear whether the leakage is confined entirely to cracks or work planes, or whether it is coming through the mass in general, due to honeycombing or gross porosity. In either case there is little likelihood that the leaks will seal themselves even though the water is heavily charged with lime.

As the inquirer has apparently sensed, because of the fact that the ground water may already be saturated with lime, there will be no further solution of lime from the concrete, but this is not the same as saying that because of the lime carried by the water the openings will be sealed by lime deposits. Except in the case of very small openings the leakage is likely to continue indefinitely. Probably the only remedy for this is pressure grouting.

Rejection of Cement which Stiffens Prematurely (39-129)*

Q—Should a cement which exhibits premature stiffening be rejected or should the type of work on which the cement is to be used govern the acceptance or rejection?

By R. F. BLANKS†

A—The answer to that question is a series of *ifs*. *If* the cement was rejected on the basis that it exhibited premature stiffening at the mill, there are ninety-nine chances out of a hundred that *if* we took it to the job, it would not cause any trouble. *If* it was rejected on the basis of whether or not it caused trouble after it got on the job, it would be too late, because the job must go on. So, *if* we could determine in advance whether or not cement was going to cause trouble on the job, and *if* the cement manufacturers could determine in advance how they might correct it, then we might be able to devise a specification provision to cover this troublesome characteristic, *if* we could devise a satisfactory specification test.

Soundness Tests of Aggregates (39-130)*

Q—Has any relationship been established between soundness tests of aggregate and resistance to freezing and thawing of concrete made with the same aggregate?

By M. O. WITHEY‡

A—Information now available indicates that the salt and freezing and thawing types of accelerated soundness tests when properly made will usually detect aggregates containing large percentages of unsound material. Aggregate particles which cause surface pitting of concrete such as shales, ochers, soft sandstones, certain argillaceous limestones, magnesian limestones free from chert and some of the lighter, more porous cherts can be detected by such tests. If the results of such tests are combined with field observations of weathered exposures of rock outcrops, much valuable evidence concerning the probable durability of these aggregates may be obtained. On the other hand if judgment were based on the results of salt and/or freezing and thawing tests of an aggregate only, many of the softer sedimentary rocks of high absorption which have given satisfactory service in well-made concrete would be rejected. The information on the significance of soundness tests of fine aggregates is less conclusive than that based on coarse aggregates. In conclusion, it is fair to say that in most instances accelerated soundness tests are likely to reveal the presence of large percentages of dele-

*ACI Quiz Session, 1942 Convention.
†Bureau of Reclamation, Denver.
‡Professor of Mechanics, The University of Wisconsin, Madison.

terious and unsound particles of aggregate but the indications of such tests should not be the sole criteria of acceptance or rejection. The most satisfactory method for ascertaining the durability of an aggregate to freezing and thawing is to include the questionable aggregate in combination with the cement and other aggregates proposed for use in carefully prepared concrete beams and then subject the beams to freezing and thawing tests. The rate of deterioration can be ascertained by measurements of the dynamic modulus of elasticity at intervals. The most quickly determined final index of damage is afforded by the loss in modulus of rupture in cross-bending. A good resume of the significance of soundness tests of concrete aggregates can be found in the ASTM Report on the Significance of Tests of Concrete and Concrete Aggregates.

Slump Limit for Vibrated Concrete (39-131)

Q—What maximum slump mixture should be permitted where concrete is to be placed by vibration? (The question, asked at the Institute's 1942 convention brought replies from Professor Withey and Mr. Meissner of "4 inches"; from F. R. McMillan: "x inches, x being the function of condition of placement and type of vibrator." Messrs Withey and Meissner subsequently elaborated their replies.—EDITOR)

By H. S. MEISSNER*

A—Vibration is a modern method of compaction. It has its greatest value when used to compact dry, low-slump mixtures, which would otherwise be placed with difficulty or else not at all. It is possible to do harm, by segregating the ingredients, if a wet mix is placed by vibratory methods. Overvibration is more likely in a wet than a dry mix. There is therefore some limit in slump, above which vibratory methods are liable to do more harm than good. In my opinion concrete having a slump greater than four inches can be easily, very easily, placed by spading, puddling and rodding and that vibration would be a dangerous adjunct. I might say that vibration methods would not be required to compact such concrete; but I might also argue that no such slump is required in the first place.

By M. O. WITHEY†

I think there is danger from segregation when concrete with a slump of more than 4 in. is subjected to internal vibration. The best rule to follow is to make the consistency of the concrete as dry as the limits of placeability will permit. Thus, in pavement construction, the best con-

*Bureau of Reclamation, Denver; chairman ACI Committee 609, "Vibration of Concrete."
†Professor of Mechanics, The University of Wisconsin, Madison.

sistencies are obtained with slumps between 1 and 2½ in.; in dam construction slumps are usually kept below 4 in., and in building construction the narrowness of the members and close spacing of the reinforcement sometimes necessitates the use of a more fluid mix. When this condition obtains, caution must be exercised to prevent segregation from overvibration.

2,000 Concrete Bath Tubs Made for a Utah Housing Project (39-132)

By OTTO BUEHNER*

For a Federal Housing project in Utah where enameled cast iron bath tubs could not be used, Otto Buehner & Co., Salt Lake City, made 2,000 concrete bath tubs in 90 days, holding designs within limits which avoided floor plan or rough plumbing changes.

We were asked to show samples of concrete made smooth enough to be considered for bath tubs in 2,000 dwellings to comprise F.P.H.A. Project Utah 42012. Enameled iron tubs having been prohibited, substitutes were considered of plywood, built-in-tile, plastics and finally concrete—lowest in price, 30 per cent less than the prohibited enameled iron.

We were asked to deliver 2,000 tubs in 90 days—22 tubs a day. There were initial delays in designing and making molds and in mix design. The final result was 40 tubs per day.

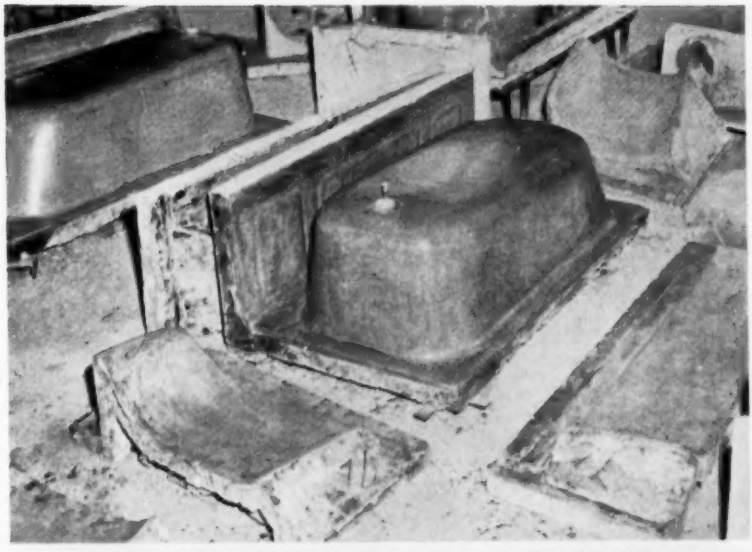
Sheet metal, wood, Masonite, rubber, plaster and plastics were all considered for molds—concrete molds were used from which a recess tub 5 ft. 0 in. long, 16 in. high and 30 in. wide could be made—conforming to conventional design to avoid costly dwelling unit changes.

Molds were polished as marble is—with grit, hone and buff. A wax coating was then applied to prevent the cast concrete tubs from adhering to the concrete molds. Molds produced a minimum of 100 casts.

The first tub suffered a good deal of shrinkage and the mix design was changed. In the course of the work five different cements had to be used, each one with its own varying characteristics. Aggregate of ⅜ in. maximum was used in a 1:3 mix, with 5 gallons of water per sack of cement. By allowing an initial shrinkage period and using both air and electric internal vibrators good results were obtained. A tub was produced from a mold each 24 hours. Tub sidewalls were 2-in. thick; reinforcing only in the rim. Each tub weighed about 700 lb.

After being removed from the concrete molds the tubs were placed in steam curing rooms. They were then buffed over with a light felt

*Member American Concrete Institute; Otto Buehner & Co., Salt Lake City, Utah.



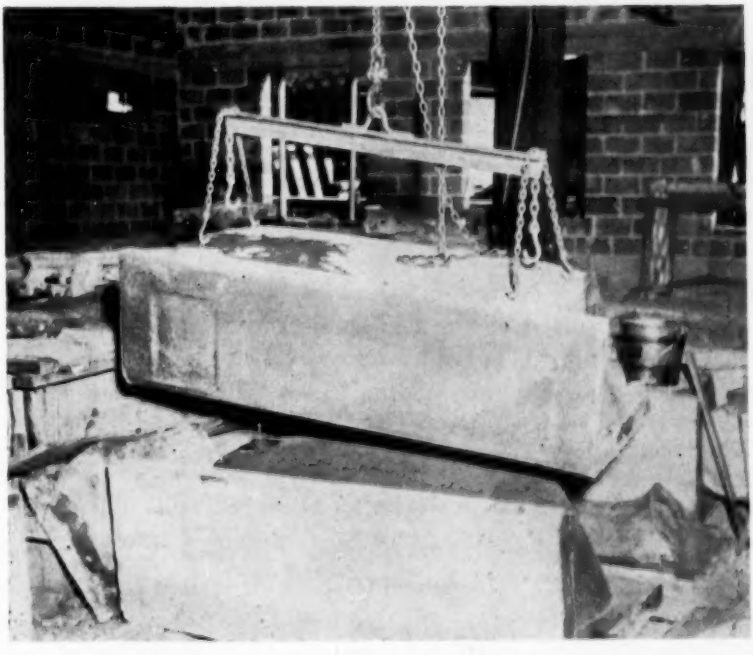


Fig. 1 (top)—Concrete mold in the foreground with the accompanying sides lying flat. On the extreme left a second mold is shown; the sheen gives an idea of the smoothness
Fig. 2 (bottom)—Tub being withdrawn from mold

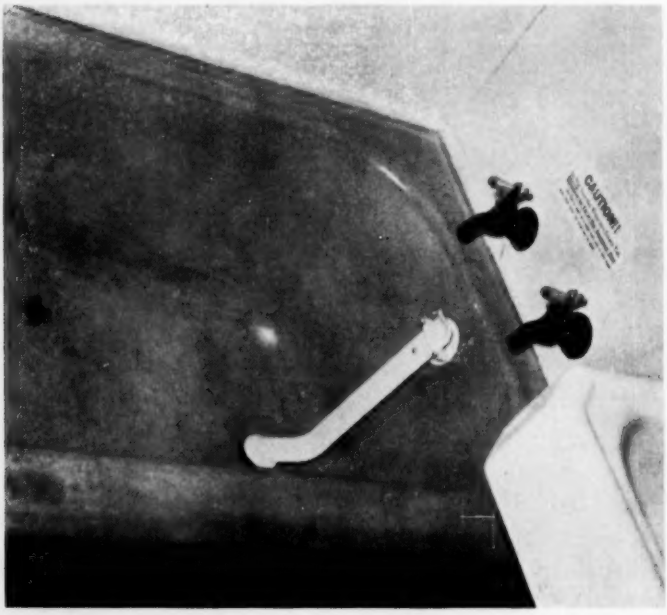


Fig. 3—Tub installed in the bathroom with hot and cold water above, and the plastic drain and overflow as the drain stopper. The "caution" is: "Do not turn hot water into empty tub. Instructions for use of war emergency tub," etc.

buffing wheel. This developed a lustre—a real gloss on the cement surface.

A plastic drain and overflow was used, in one unit, held in place at the bottom by saw tooth points projecting into the drain hole and the top of the glider supported at the highest desired water level by a guide attached to the tub. The glider stem is hollow and is the overflow through which the water flows down into the drain. It is of a light ivory color.

Steam curing gave the tubs a light, uniform, gray color. When made of white cement concrete tubs take on an appearance difficult to distinguish from the white porcelain tub at a distance of 5 to 6 ft.

The precast concrete tubs have now been in service for six months. Reports at frequent intervals are that users like them, because easily kept clean with a soapy cloth and water and the appearance of the tub improves with use. A notice on each tub as installed advises users not to turn hot water directly into an empty tub. No cracks have developed from thermal shock.

How High a Lift of Concrete at One Placing (39-108)

By F. L. FITZPATRICK*

I think that the discussion of the maximum lift of concrete at one placing, September 1942 JOURNAL, neglects one very important factor.

A corollary to the bleeding or water gain is the slumping of the concrete after placing, and before any appreciable hardening takes place.

In concrete with horizontal reinforcement, where the depth of pour, in relation to the thickenss of the section, is considerable, this slumping has a definite tendency to cause cracking due to the concrete below the reinforcing bar sagging away from that immediately above the bar, because of the restraining effect of the bar.

Mention is made of a ten year life period proving that no damage ensues. I have often observed, in inspecting old work, that cracks develop along the line of horizontal bars, though the concrete may stand for years, without such cracks being apparent.

Even when no crack is visible at first, there is apparently a plane of weakness, and no doubt the movements in the concrete due to temperature and moisture change, tend to localize cracking at any such weak planes.

Incidental to the same thing is the well known fact that bond on the under-side of horizontal reinforcing bars is generally poorer than on the upper 180 degrees, a considerable weakening in bond value will tend to result with very high pours.

I think that it is important, therefore, in considering the matter of limiting lifts in pouring concrete, to keep well in mind the dangers of the cracks and lowered bond values. A close examination of old concrete structures will reveal the importance of this matter, and how neglect of it contributes to shortening the life of the structure.

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Current Reviews

of Significant Contributions in Foreign and Domestic Publications, prepared by the Institute's Reviewers

Concrete reservoirs of the vertical-beam type

C. MAXWELL STANLEY, JR. *Proc. Am. Soc. C. E.*, V. 68, No. 10, Dec., 1942, p. 1679-1688.
Reviewed by H. J. GILKEY

Reinforced concrete walls designed as vertical beams spanning from a hinged type of connection at the base to roof or tie rod arrangement providing another hinge connection at the top. With the sidewalls loaded as simple beams the inside face is in compression which tends to prevent horizontal crack formation. The design details are described and illustrated and service data are given on three such reservoirs. Some of the details were altered after the construction of the first reservoir. A number of alternatives are possible with respect to joints, connections, etc. Author discusses and compares his designs with other conventional types, citing both the relative advantages and disadvantages and indicating some of the limitations of his own and other designs.

A study of chert as a deleterious constituent in aggregates

HAROLD S. SWEET and K. B. WOODS, Bulletin, Research Series 86, Engineering Experiment Station, Purdue University, Lafayette, Ind., 110 pages. AUTHOR'S SYNOPSIS

This report covers the study of chert as a deleterious constituent in concrete aggregates conducted as a co-operative investigation between the Bureau of Materials and Tests, State Highway Commission of Indiana, and the Joint Highway Research Project.

The investigation was divided into two parts: Indiana limestone quarry cherts and Indiana gravel cherts. The various definitions of chert and the theories concerning its mode of formation were reviewed. The work of previous investigators was studied and their conclusions were reported.

Representative samples of chert from practically all producers of stone and gravel aggregates in Indiana were secured, together with some materials from surrounding states. Ledge cherts were obtained from quarry faces and identified geologically. A record was made of the visual appearance of each sample, including color, luster, texture, and fracture. Each sample was subjected to performance and identification tests. Performance was determined by imbedding evacuated and saturated chert in mortar cubes and subjecting the cubes to alternate cycles of freezing and thawing. Identification tests consisted of specific gravity, absorption, degree of saturation, dye penetration, unconfined freezing and thawing, chemical analyses, and microscopic examination of thin sections. Some of the quarry chert types which were non-durable in the performance tests were investigated further to determine the effect of dry freezing and thawing and of alternations of temperature above freezing. The influence of particle size on the durability of

these types was tested by freezing and thawing mortar cubes containing chert specimens of various sizes, saturated with water. Another series of tests was conducted to determine the effect of alkali on Indiana chert.

The same general performance test procedure employed on the quarry cherts was used to determine the durability of the gravel cherts. The gravel chert specimens that failed were removed from the mortar and subjected to the following identification tests: color, texture, bulk specific gravity by the flotation procedure, dye penetration, apparent specific gravity, absorption, degree of saturation, and microscopic analysis. The non-failing specimens were removed from the freezing and thawing test at the end of 40 or 160 cycles, broken from the cubes, and then analyzed, using the same identification tests as were used with the failures. A series of mortar tests was conducted to determine the difference in performance of river-wet, evacuated and saturated, and immersed gravel chert. The absorption characteristics and the effect of stockpiling of river-soaked gravel were studied. A limited study of the effect of stain on absorption characteristics of gravel cherts was made. To furnish a comparison with chert, specimens of limestone, sandstone, shale, ocher, igneous rock, and quartzite were imbedded in mortar cubes and then frozen and thawed.

The bulletin contains a review of the literature on chert including a 45-reference bibliography. A total of 23 figures and 21 tables represent the data obtained. The 34 important results are summarized at the end of the report with a listing of eight major conclusions.

Bond strength of reinforcing bars embedded horizontally in concrete

R. C. Robin, P. E. Olsen and R. F. Kinnane. The Journal of the Institution of Engineers Australia, V. 14, No. 9, Sept. 1942, p. 201. Highway Research Abstracts

This paper reports results of preliminary tests made with the purpose of discovering what factors influence the bond of bars cast in the horizontal position. So many factors have been found that the preliminary tests grew into an extended series conducted over three years. Although incomplete, the results of the investigation to date have appeared to be of sufficient interest to record in a preliminary report.

In order to avoid the introduction of too many variables, most of the work has been done with two classes of concrete: Class (A) 400 psi., and Class (B) 2000 psi.

Method of casting. The blocks were cast in moulds resting on paper on a cement floor, in the same way as the ordinary compression test cylinder. The bar passed through the moulds and was thus prevented from sinking with the concrete. In a few cases, however, the bar was so arranged that it could be released immediately after the concrete had been poured and left floating in the wet concrete. Where bars passed through one wall only of the mould, as in the case of hooked bars, the projecting end was clamped, to avoid any possibility of vibrating movement during tamping.

Method of curing. The pull-out test specimens were stripped two days after casting and cured under water until testing. Test was at age 28 days for ordinary cement, and at 12 days for those specimens in which rapid hardening cement was used.

Method of testing. Most of the tests were made with a spherical bearing block introduced between the concrete test block and the top platen of the testing machine. The measuring dials were so arranged that both the end and bottom slip of the bars could be determined during the test.

Conclusions. The following tentative conclusions have been drawn from the tests carried out to date: (1) For bars placed up to a height of 7½ in. above the bottom of the block (the limit of the experimental work), bond strengths of fixed horizontal plain bars drop off rapidly as height above the bottom of the pour is increased. At the limiting

height used (7½ in.), bond strengths obtained from pull-out tests were in many cases lower than the so-called safe working stress of 0.04 f'_c. (2) Straight bars near the top of a beam, which are restrained from sinking with the concrete as it settles, may develop little bond and should not be used as top reinforcement in continuous and restrained beams. (3) Straight bars freed from restraint and allowed to settle with the concrete, develop good bond. (4) Hooked bars used as top reinforcement in beams, have developed good values for ultimate strength. Such bars rigidly held against vertical settlement, may develop such an abnormally large slip before the anchorage comes into play that it is questionable whether they carry any stress at ordinary loads. (5) A straight bar in a favorable position, such as at the bottom of a pour, or in a vertical position, is at least equal in bond strength to a hooked bar in a narrow block. (6) The working stress in bond should not be increased in proportion to the crushing strength of the concrete. (7) The working stress (allowed under the S.A.A. Building Code) for bars with hooked ends, namely, twice the stress for straight ended bars, is far higher than is justified. (8) Embracing hooks or turned bars develop the yield strength of the bar, but if closely spaced, may split the concrete. (9) The practice of allowing increased working stresses in bond for end anchorages or anchorages beyond the points of inflection in beams, is to be deprecated, since large deflections must occur before such anchorages come into play. (10) Rigidly held horizontal reinforcement in construction is a source of weakness. Such reinforcement not only may have weakness in bond, but it may cause longitudinal cracks in the surface above the reinforcement and may facilitate corrosion of reinforcement on account of the small "settlement channel" which forms under the bars. (11) External vibration of specimens gave a slight improvement to the bond for top bars in the case of 2,400 lb. concrete, and a marked improvement in the case of 4,000 lb. concrete. (12) Pull-out tests of the deformed bar used, showed that this bar develops the yield strength of the bar when cast into thick blocks, but only after large slip. Cast near the top of thin blocks, this type of anchorage gave decidedly higher ultimate strengths than did hooked plain bars. (13) The effect of making the moulds completely water-tight was to decrease the bond strength of bars cast near the bottom and to increase, somewhat, that of bars cast near the top of pull-out blocks and beams. The data in this report are presented in eleven figures and nineteen tables.

Camouflage paints and their application to road and building surfaces

L. G. Gabriel. (Paper before Road and Building Materials Group of Society of Chemical Industry, Great Britain, Oct. 6, 1942.) *Chemistry and Industry*, V. 62, No. 1, Jan. 2, 1943, p. 3.
Highway Research Abstracts

The primary object of the general run of camouflage schemes is to confuse the pilot of a bombing aircraft for a period which may be as short as a fraction of a second at the moment when he is about to release his bombs. Camouflage paints are fundamentally the same as normal paints used in peace for decorative and protective purposes. The general characteristics desirable in camouflage paints are that they include provisions for the opacity, color, and finish of the paint, the surface characteristics, resistance to water and alkali, and behavior under an accelerated weathering test. The opacity requirements are very low compared with those for peace time paints, the object being to develop an adequate camouflage effect with the minimum consumption of raw materials.

Camouflage painting of road surfaces. The preparation of road surfaces for painting is of the most crucial importance and a very large proportion of the difficulty and trouble which has been encountered—particularly with the camouflage of runway and dispersal point surfaces on aerodromes—must be attributed to the fact that reasonable precautions have not been taken with the application of the paint.

Many experiments before the war in producing colored highways mainly for aesthetic purposes failed to yield a complete solution to the problem of coloring road surfaces carrying heavy traffic. Since this problem has again been tackled for camouflage purposes it has become evident that broadly speaking road surfaces had to be divided into two main classes from the point of view of painting. For convenience these classes may be styled (a) tar and bitumen roads, and (b) concrete roads. Most tar and bitumen roads have somewhat rugous surfaces, and although when the paint is applied it naturally covers the summits of the chippings as well as the declivities, it is evident that the traffic wear will be very largely on the summits of the chippings. The paint, therefore, remains in the declivities and produces its tonal effect for a considerable period. There will in many cases be discoloration due to bleeding through of the original bituminous or tar binder, and also by oil and dirt droppings from the traffic itself, but experience has shown that such roads when painted with bituminous emulsion paints retain a good camouflage effect for considerable periods, depending on the amount of traffic.

Concrete roads. Concrete roads form a completely different picture; quite apart from the fact that the concrete itself may show wide variations of texture, the essential problem is different. The surface is not sensitive to the penetration of spirit solvents or oils, but, on the other hand, it is extremely hard and unyielding and the position is often complicated by the presence of laitance or other dusty deposits. The problem is, in fact, to secure good adhesion of the paint film to the concrete surface, and secondly, to have extremely good resistance to attrition. Much work has been done to develop special types of paint for these purposes, and the efforts of the research departments of individual paint firms coupled with the work carried out at the Paint Research Station at Teddington have yielded encouraging results. The general opinion is that certain spirit types of paint have an intrinsically high performance, and for the painting of concrete roads these spirit paints may be regarded as a satisfactory solution of the problem. Bitumen emulsion paints have also been shown to give very good results, even under moderately heavy traffic, but for this purpose it is necessary to apply them rather heavily, say, at about eight yards to the gallon. Suitable spirit varnish types of paint can be made to give good results at a covering capacity of about 25 yards to the gallon. When employing bituminous emulsion paints at the recommended rate of coverage they will normally be applied in two coats, as it is difficult to hold the requisite amount of paint on the surface in one application.

Painting technique on roads. A good deal depends on the method of application adopted in painting roads of any type if the maximum life of the paint is to be secured. Spraying and brushing separately do not seem to give such good results as a combination of the two; it is a very curious fact, particularly in bitumen emulsion paints that the maximum bind of the bituminous medium on the pigment particles is not obtained in the early stages, at any rate, if the paint is simply sprayed; this must be associated with the deforming action of the paint brush on the bitumen particles in the deposited paint film. Brushing alone on large horizontal surfaces is rather a cumbersome method and does not give the same degree of finish as spraying. The best result seems to be achieved by spraying from any ordinary type of spray gun, or even better with a road sprayer of the type used for surface dressing, followed by brushing with a medium stiff broom. The men wielding the brushes can follow immediately behing the sprayer, and except for the necessity for a slight increase in the labor force carrying out the operation, there is no great increase in expense, and the rate of progress is equal to that obtained by spraying alone. It is highly advisable to adopt this technique in painting any sort of road surface, and it is particularly essential with concrete roads. Other phases of the problem are discussed.

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The Properties and Behavior Underwater of Plastic Concrete*

By P. J. HALLORAN† and K. H. TALBOT‡
Members American Concrete Institute

SYNOPSIS

This paper pioneers in the field of analyses of the physical characteristics of plastic concrete when placed under water. No endeavor is made to record the field procedures except insofar as they are related to the research as set up by observations, tests in the prototype, and in models. The conclusions are consistent with the limited number of repetitive readings. It suggests by observation of certain basic principles that greatly improved physical characteristics of concrete may be developed under water by modifying the field procedures to accommodate the indicated behavior from the numerous experiments.

It is the hope of the authors that observation of several of these principles which appear to extend the heretofore limited knowledge of this field of engineering may lead to continued use of tremie concrete for engineering problems that require accurate and intricate placing of a dense, impermeable, durable concrete.

INTRODUCTION

The engineering field has heretofore considered tremie concrete as a medium for the accomplishment of specific and unusually difficult phases of under-water construction. Such concrete has been used primarily as a seal or a foundation. However, with the development of the need for rapid, economical and efficient construction of dry docks in great numbers, Rear Admiral F. R. Harris (CEC) USN (Ret.) with great foresight and courage, conceived the possibility of adopting tremie concrete for the construction of these large basins, in a manner that would effectively unite steel and concrete to form structural members under water.

For those of us in the field there were but few rules, since concrete in this form had seldom or never been used with structural value. There

*Received by the Institute Jan. 18, 1943.
†Captain (CEC) U. S. Navy.
‡Concrete engineer.

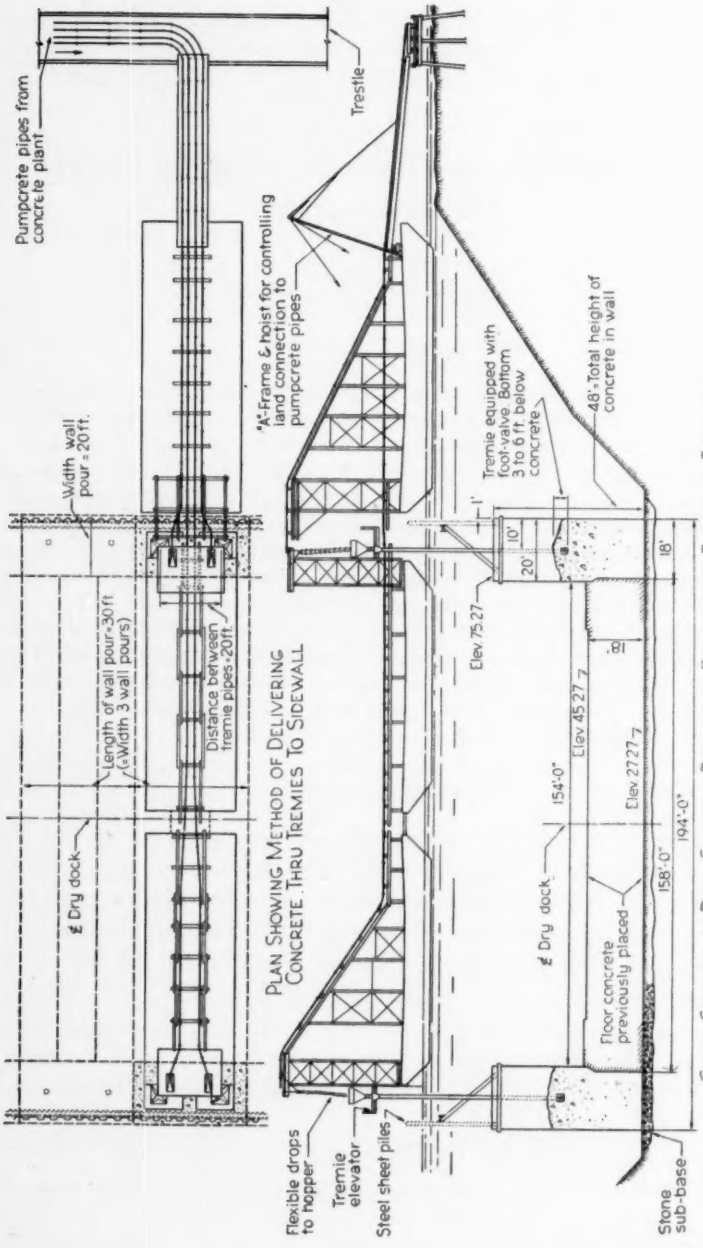

Pumpcrete pipes from concrete plant
Trestle
"A"-Frame & hoist for controlling land connection to pumpcrete pipes
Width wall pour = 20 ft.
Length of wall pour = 30 ft (=Width 3 wall pours)
Distance between tremie pipes = 20 ft
℄ Dry dock
PLAN SHOWING METHOD OF DELIVERING CONCRETE THRU TREMIES TO SIDEWALL
Tremie equipped with foot-valve Bottom 3 to 6 ft below concrete
48'-Total height of concrete in wall
Elev. 75.27
Elev. 45.27
Elev. 2.27
154'-0"
194'-0"
158'-0"
Floor concrete previously placed
Flexible drops to hopper
Tremie elevator
Steel sheet piles
Stone sub-base
CROSS SECTION OF DOCK SHOWING PLACING OF CONCRETE BY TREMIE IN SIDEWALL

Fig. 1—Schematic drawing showing installation of tremie and tremie barges for a dry dock.

were no standards; only the briefest sort of a specification. Thus it became necessary for all of those connected with the administration of the construction in the field to compare notes with the experiences on other docks, and to endeavor to record, as well as practicable without interference with the work, the characteristics of tremie concrete placed under entirely different conditions and ultimate use.

Investigations were carried on under the sponsorship of the Bureau of Yards and Docks under the direction of the writers, during the construction of a dock. It will be the endeavor to convey to engineers the experience of those who were intimately connected with the placing of concrete, believing that others will doubtless find greater and more extensive use for this method of placing. It is appreciated that these recordings deviate somewhat from what had been formerly expected. They have been checked by observations on other structures. The paper that follows will strive to describe and record the findings of these investigations—referring not only to their quantitative value, but bringing out features which the writers consider to be relatively new, even entirely new, on the plastic flow of concrete materials in deep water.

It is now appreciated that the flow characteristics of tremie-placed concrete and the flow pattern of the material as extruded from the bottom of the pipe tremie, as well as the type of delivering equipment, will have an effect on form design and on construction practices. If, in planning a project, concrete is to be delivered by large concrete buckets discharging into tremies without control hoppers, large tremies and high instantaneous velocities will result, with deleterious turmoil within the already placed concrete as the tremies are charged. If the concrete is delivered to the tremie at a more uniform rate from a truck mixer, the size of the tremie and the amount of the disturbance can be reduced. Both of these systems have the disadvantage that intermittent delivery of batches of concrete results in setting up new flow lines through the mass and breaking out new flow channels, together with the possibility of cleavage planes in the finished product.

Only by smooth continuous flow of concrete to and through the tremie can uniform distribution be assured. Creating flow after the concrete has partially stiffened is difficult. The tendency can best be overcome by the adoption of continuous rapid placing. Exact information on many characteristics of under-water flow was not available to those responsible for the design and construction of the dry docks. It soon became apparent that there was need for more extensive knowledge of the characteristics of the flow of the extruded concrete into and through the body of the already placed plastic concrete. Only by definite knowledge and the application of these observations could a field procedure be

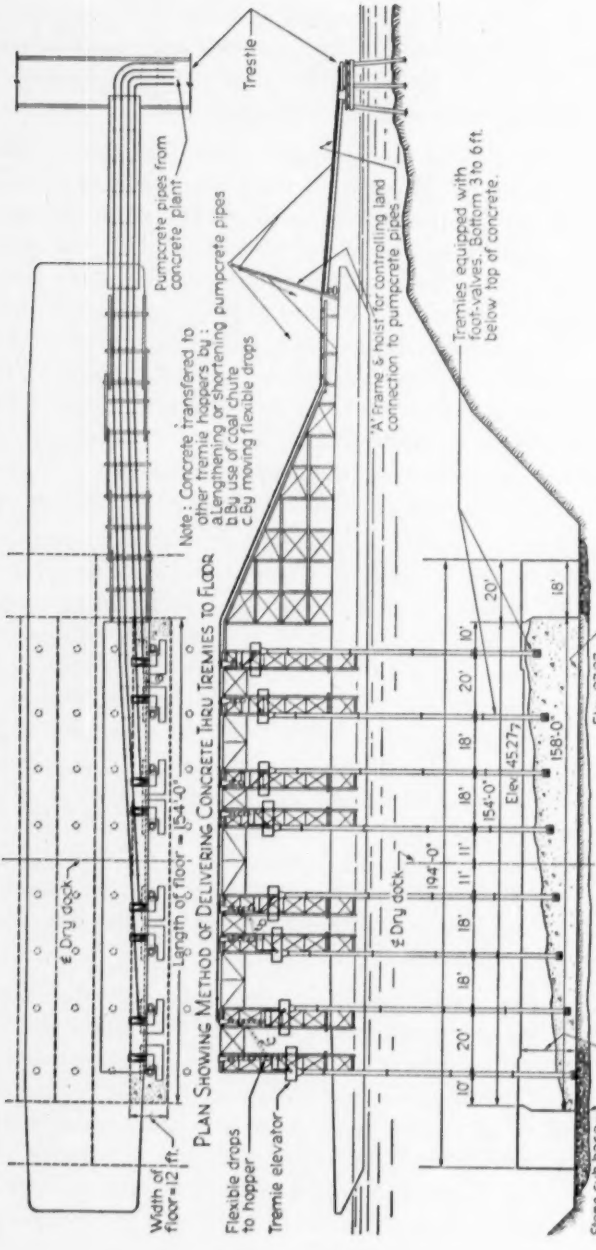

Trestle
Pumpcrete pipes from concrete plant
Note: Concrete transfered to other tremie hoppers by:
a Lengthening or shortening pumpcrete pipes
b By use of coal chute
c By moving flexible drops
"A" frame & hoist for controlling land connection to pumpcrete pipes
Tremies equipped with foot-valves. Bottom 3 to 6 ft. below top of concrete.
℄ Dry dock
Length of floor = 154'-0"
Width of floor = 12 ft.
PLAN SHOWING METHOD OF DELIVERING CONCRETE THRU TREMIES TO FLOOR
Flexible drops to hopper
Tremie elevator
℄ Dry dock
194'-0"
154'-0"
158'-0"
Elev. 45.27
Elev. 27.27
18' - Total height of concrete in floor
Stone sub-base
CROSS SECTION OF DOCK SHOWING PLACING OF CONCRETE BY TREMIE IN FLOOR

Fig. 2—Schematic drawing showing installation of tremies and tremie barges for floor pour

developed that would result in concrete of maximum uniformity throughout the finished structure. Knowledge of the direction and intensity of pressures was required if form design was to be adequate and at the same time economical. Differences between designing and construction engineers needed to be aired to develop field controls that would give the desired results.

THE PROBLEM

Reduced to lowest terms, the problem confronting the field consisted of the preparation of a site, and the building of a structure having 250,000 cu. yds. of concrete, of which 196,000 cu. yds. were placed under water. The structure has these dimensions: overall length, 1,150 ft., width, 194 ft., height, 72 ft. When unwatered, it was to be ready to receive the wall lining, the finished floor and the super-structure concrete. The tremie-placed floor of the dock was 18 ft. thick and the walls 20 ft. thick. All of this concrete was to be placed, and the outboard end of the dock closed before any of the concrete could be seen or inspected except within the limitations of the divers. Success in such a project required the separation of the project into operations that could be expeditiously and economically repeated. This called for engineering, design, and field planning, so that the construction could be carried on under water without visual inspection, and with only such inspection by "feel" as was possible by the divers. Visualization of each step required that it be so clearly outlined that all construction and supervisory personnel might understand their duties. Drawings, models, and charts were devised. Men were trained in methods of doing each part of the work, for minute details not only had to be developed in the drafting room, but had to be executed under water in the field.

The steps preliminary to the placing of the concrete can only be mentioned in this paper. The completeness with which each of these was carried out and the thought required in their execution justifies a separate presentation. Suffice it to say that each of these steps had to be planned to fit in with the placing of the concrete. Following the removal of the excess excavation by a large dipper dredge equipped with a 90 ft. dipper stick, it became apparent that light silt and water-borne mud must be picked up by a hydraulic dredge, to reduce the redepositing of the material on parts of the work from which it would have to be again picked up. The stabilization of the subgrade by a gravel or stone blanket developed the problem of striking off this surface of the gravel to receive the concrete forms. During the driving of piles, this gravel surface must not be disturbed. Further, the "hold down" piles must be driven to exact line so that the portion extending above the blanket would not

interfere with the reinforcing steel carried by the form trusses as they were lowered.

Truss forms weighing as much as 176 tons required careful and exact fabrication, installation, and placing, so that when placed in 70 ft. of water, bulkhead doors could be closed and locked. The exact placing of the forms required careful detailing of special handling equipment. The forms had to be placed and sealed against the gravel bed in such a manner that the concrete would not leak from one form to the adjacent space. The concrete plant, the distributing system, and the tremie barges had to be planned, purchased, and erected. The procurement of special floating equipment including car floats, tremie barges, derricks, tugs, and divers' scows, formed part of the problem.

Each of these was a major construction operation. However, they were only preliminary steps to the main project of placing concrete under water with tremies so that the dry dock would successfully fulfill its mission of furnishing a dry erection basin on which the largest naval ships might be built on an even keel and floated without being launched from ways.

For convenience and control of cracks, the floor of the dry dock was divided into sections running transversely to the dock. These sections were 154 ft. long, 12 ft. wide, and 18 ft. thick. The wall pours were divided into sections 20 ft. wide, 36 ft. long, and 48 ft. high. The concrete in the floor section was placed in one continuous pour between side wall forms, and the side walls were poured full depth to bottom of floor sections. The floor trusses to act as concrete forms were fabricated from structural steel with an overall length of 194 ft., extending from out to out of the walls. Bulkheads separated the floor from the wall sections. The floor trusses were set 24 ft. on centers bringing the ends of the wall forms in close contact and leaving a 12 ft. intermediate space between floor forms to be subsequently filled with concrete. The forms carried the reinforcing steel for the section and for half of the intermediate section, as well as the bottom steel for the walls. Forms were equipped with sight towers having a height such that when the form was on the bottom, the sight towers could be seen above water and the location of the form thus determined. The forms were sealed by equipping them with a roll or bolster of expanded metal, sufficiently flexible to take up the irregularities in elevation of the gravel blanket. This sealing of the forms proved to be an important factor in the success of the operation, for if this seal between the form and the rock blanket positively stopped the leaking of concrete under the forms, the concrete could be placed as rapidly as desired.

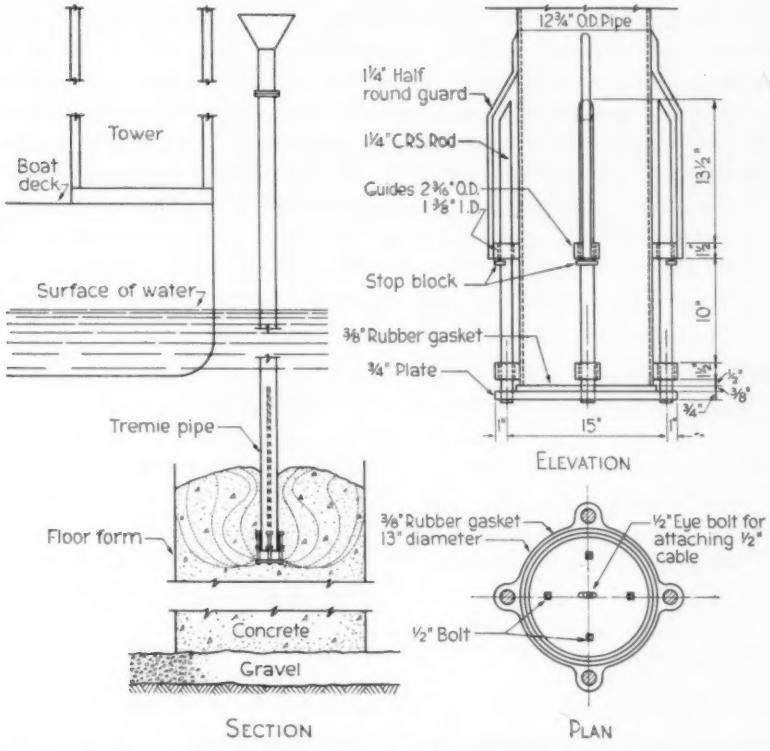


Fig. 3—Detail of tremie foot valve in plan and elevation (foot valve closed) and (foot valve open) as used in placing concrete, showing distribution of concrete under water

PLACING METHODS COMPARED

Whereas, four yard buckets delivered concrete directly to hoppers above tremies for one dock, truck mixers delivered into the tremies for another, and belt distribution was used for still another dock built in the dry. Pumperete delivery was adopted for the dock here considered. Continuous delivery through concrete lines simplified the problem of distribution. Not only was it easier to maintain constant flow from the bottom of the tremie, but it was much simpler to feed the tremies from a top deck than would have been the case had buckets been adopted.

The development of a plate foot valve placed 11 in. from the end of the tremie allowed the distribution of concrete sidewise and upward without disturbing the rock bed or the concrete below the bottom of the tremie. This valve, held by a cable when closed, allowed the unwatering of the tremie; and when opened controlled the distribution of the concrete. It was believed that a 10-in. tremie would have been preferable to

the 12-in. tremie specified and adopted. Where open-end tremies were used, the concrete delivered at a rate of from 50 to 60 yards per hour, the 12-in. tremie proved to be unnecessarily large. The most desirable spacing of the tremies depends to a considerable extent on the continuity with which the concrete is delivered. In general, it is believed that the spacing should be reduced from 19 to 20 ft. to 12 to 16 ft., and that the end tremies should be placed less than 9 ft. from the bulkhead. Although four pumpcrete lines delivered concrete to eight tremies, the experience gained leads to the belief that it might be practicable to arrange the plant to deliver concrete continuously by connecting the pumpcrete lines directly to the tremie pipe.

The delivery of concrete to the wall sections of two docks showed the advantage to be gained by reduced spacing of the tremies. In one, the section was 36 ft. long and 20 ft. wide; in the other 42 ft. long and 20 ft. wide. Two pipes 20 ft. apart furnished the concrete to the first, and three pipes 10 ft. apart, to the second. In the first case, one pumpcrete line fed each tremie, while in the other, two pumpcrete lines delivered concrete to a hopper that discharged into three tremies. The capacity of the delivering equipment was the same. However, the surface of the concrete as shown by soundings was much smoother when tremies were more closely spaced.

The advantage of rapid placing of concrete was apparent when in finishing a floor section it was possible to deliver concrete from two pumpcrete lines to a single tremie. The velocity in this 12-in. pipe served by a single pump is only 30 ft. per minute. When this velocity was doubled, the advantage of the added kinetic energy of flow in carrying the concrete to the forms and around structural steel members was apparent.

In laying out the concrete and tremie plants, the relationship of the number of tremies to the number of pumps was largely a problem of balancing available plant, rather than judgment based on knowledge of flow. The elimination of particle interference causing resistance to flow, both during pumping and during placing, was extremely desirable. Preliminary studies in the design mix indicated that concrete made with a smooth, well rounded gravel and having a maximum size of 2-in. was superior in workability to either a concrete made with small gravel or with crushed stone having sharp angular edges and surfaces. Elongated or over-sized gravel might so nest in the pipe as to cause plugging of the tremie. The gravel was delivered in one size only and a grading was selected that would not be objectionable even with some segregation. In other words, the grading was on the coarse rather than on the fine side. A sand-gravel ratio was predetermined to assure maintaining great workability, even with some variation in gravel grading. From time to

time adjustment was made of the sand-gravel ratio to give better workability.

The cement specified was moderate heat cement passing Federal Specification SSC206A, but due to the inability of the mill nearby to deliver the required amount of cement, at a reasonable temperature, it was found necessary to accept standard cement shipped from Pennsylvania, manufactured to meet Federal Specification SSC191A. The temperature of the cement and the temperature of the concrete proved vitally important factors in the workability of the concrete. The high temperatures resulted in rapid stiffening and quick hardening, causing difficulty in placing and necessitating speeding up the operation. The proportioning of the mix was in accordance with Navy Specification 13YC, Class F2, except that an adjustment was made in the ratio of sand and gravel. Seven sacks per cubic yard of concrete and a water-cement ratio of 5½ gal. per sack were found to give a mix of desired characteristics. The resultant slump was 7 in. The concrete had not only slump, but it had plasticity, flowability, and cohesiveness. Separation was not entirely avoided, particularly as the concrete discharged from the end of the pumpcrete pipe tended to separate and segregate as it fell through the hopper and the elephant trunks. Reuniting of the aggregates took place to a large extent in the tremie and was promoted by the extrusion already described.

OBSERVATIONS AND TESTS

Cylinder tests showed the concrete to have a strength well above 4,000 lb. Cores from floor and wall sections showed weights of 152 lb. per cu. ft., great uniformity of aggregate distribution, and lack of water pockets or water films under the aggregates. This density and uniformity insured durability. Preliminary runs were made by placing concrete through tremies in test sections under water. The size of these specimens was determined by the lifting capacity of the floating equipment—for unless the block could be raised and inspected, little would be gained by flowing the concrete into place through a tremie.

Investigations developed certain fundamental information. The method of separating the sea water from the concrete in the tremie by equipping the bottom with a foot valve proved entirely satisfactory. This valve was held in closed position by a cable and steam lift while the tremie was unwatered and was filled with concrete. When the foot valve was opened, the concrete forced its way out between the periphery of the bottom of the pipe and the plate, and moving outward without disturbance formed an excellent seal. This mass under the pressure from the concrete in the tremie moved outward until it reached the side of the

form. As the rate of flow of the first concrete was controlled by the opening between the valve and the pipe, there was no tendency for the flow to gouge out the gravel bed. As soon as the bottom of the pipe tremie was covered with concrete, the flow became an extrusion from within and through the mass of concrete and not a flow over the top of the already placed concrete. This was substantiated by the test blocks in which mud, boards, and even rocks were picked up from the stone bed and carried to the top of the box by the flow of the concrete. From these preliminary tests it became apparent that best control of the concrete could be obtained if the flow characteristics were determined.

Beginning in May, 1941, on this dock project and continuing into the late months of 1942 on another project, these flow characteristics were studied and charted. When placing the first tremie concrete in the forms it became evident that the flowability of the concrete, the continuity of pumping and the rate of delivering concrete, determined the depth at which the tremie could best be held in the concrete. After the seal was formed and concrete was built up around the tremie pipe 3 to 5 ft., the concrete was forced to the side of the form with the surface slope reasonably flat. When the tremie extended into the concrete only one or two feet, the slope of the top of the concrete was steeper. The concrete in the latter case moved up along the pipe and out to the form by mass movement or by flow rather than by being extruded through the mass from the end of the pipe.

The advantage to be gained by the use of the foot valve is the elimination of the retempering of the concrete below the tremie by the force of the flow. With a vertical rise of from two to three feet per hour, the concrete opposite the bottom of a tremie with an embedment of 6 ft., will be 2 or 3 hours old. If an open end tremie is used, it is probable that the disturbance extends for another 2 ft. which might add an hour to the age of the concrete.

MOVEMENT OF CONCRETE UNDER WATER FROM TREMIE AND POSSIBILITIES OF BAFFLES

The conception of the sphere of pressures developed from the model studies has a definite application on the distribution of the concrete as observed in the field. When the tremie is carried deep the sphere of pressures is larger and the surface slopes are flatter. When the bottom of the tremie is shallow the sphere of influence is small and the slopes are sharper. Further, when the sphere is small the extruding pressures force the concrete more nearly upwards and without the resistance of a mass of concrete already placed adjacent to the pipe, it flows along the top surface of the concrete already in place, instead of being forced out from below. In summation, the shallow tremie has a distinct advantage

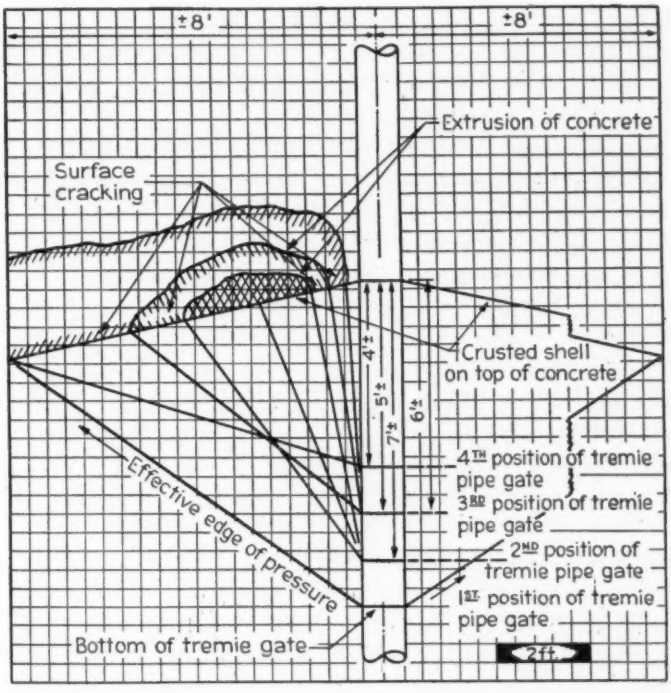


Fig. 4—Vertical half section sketch to show procedure for breaking hardened shell formed on the surface of the concrete during placing, caused by delays in placing or by rapid hardening. The condition was more often experienced when tremies were kept at a maximum depth below the concrete surface

of minimum disturbance of the concrete in place, while the deeper tremie has the superiority of forcing the plastic mass through greater horizontal distances and flatter surface slopes.

There is an opportunity to develop a tremie equipped with foot plate and baffles to control the flow, thereby allowing the distribution of the concrete with shallow tremie depth which appears to be very desirable. Some tests were made but details were not perfected. In the development of such baffles, the design should be such that there is a minimum of friction due to the change of direction and that the concrete as finally discharged from the top of the baffle will move horizontally. Only by the continuation of the use of the foot valve is it possible to deliver concrete without subjecting the concrete below the bottom of the tremie to constant retempering.

The problem confronting the designer of a practical tremie equipped with foot valve and baffle may be stated as follows: To develop a pipe tremie equipped with a foot valve and baffle which will change the direction of flow of a column of concrete effectively from vertical to horizontal

and will deliver controlled concrete where required to a maximum horizontal distance with a minimum vertical movement. To say this in another manner—to distribute concrete through the greatest possible horizontal circle with a minimum depth of tremie.

The column of concrete to be directed may have a height of over 60 ft., a diameter of from 10 to 12 in., may be moving at a rate of from 30 to 50 ft. a minute, and may be discharged under a pressure of 9,000 lb. per sq. ft. against a hydraulic head of 4,000 lb. per sq. ft., plus a negative concrete pressure due to the head of concrete already placed, of from 80 to 300 lb. per sq. ft. The design should be such as to direct the flow with a minimum of head loss. The discharge at the outer periphery of both the baffle and the valve must be horizontal although the friction of discharge may be reduced by giving both foot valve and baffle deflectors a slight roll at the edge.

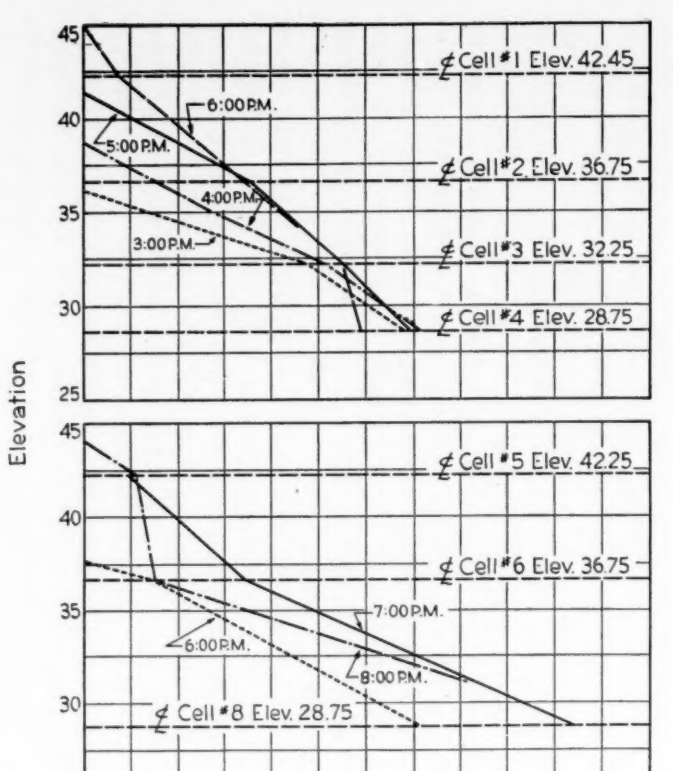
Observations of concrete as placed in the forms showed that the rate of hardening was faster than had been expected. It had been assumed that the concrete remained fully liquid for as much as five hours, and that the concrete adjacent to the tremie pipes remained fluid for a longer period than that adjacent to the walls. It soon developed that instead of the concrete remaining plastic for five hours, the surface of undisturbed concrete lost plasticity within 45 minutes. Thus, it became essential that the concrete be placed in final position as quickly as possible by continuity of flow. The method adopted on this project was to maintain a relatively uniform depth of tremie, raising it about one foot at a time as the elevation of the dome of the concrete rose. If the tremies were carried too shallow, the surface slopes were steep. The mass at the forms and at a distance from the tremie had a tendency to harden rapidly, whereas the concrete at the tremie remained longer in plastic condition, due to agitation. On the other hand, if the tremie was too deep, there was a tendency for a hard shell to form, extending from the tremie outward toward the forms. However, when the tremie was carried at the proper depth (between three to five ft. deep) and the placing of the concrete was continuous, this shell did not form. Under these conditions, there was a constant building up of the surface over the whole area of the form and newer concrete was forced into place so that the concrete lower down was not disturbed longer than was necessary. By careful control of the depth of the tremie it was entirely feasible to eliminate objectionable surface hardening.

With constant pumping of uniform concrete, it was possible to carry the tremies in the concrete to the desired depth and to raise them 6 inches or a foot at a time. Few delays resulted from such tremie operation. If pumping was intermittent or the concrete was harsh, the depth of the tremie was reduced.

The rate of placing concrete was affected by the number of pumps and pipe lines available, the condition of the pumping equipment, length of line, and pumpability of the concrete. Pipe stoppages caused expensive delay. As the work progressed, closer coordination of the plant and barge organizations reduced loss of time. When a pump commenced to limp, the barge foreman was advised of the possibility of a shut down and he was able to make pipe adjustments before it became necessary to take the weak pump off the line. If the barge foreman gave the plant warning before he cut off flow in one line, arrangements might be made for adjustments that would allow removal of equipment requiring maintenance rather than the shutting down of a pump in good repair. Even short delays caused a stoppage of the flow which must be again started either by increasing the head of concrete in the tremie or by raising the tremie sufficiently to allow the concrete to develop new flow channels. The maximum elapsed time between the delivery of concrete to a tremie was 10 minutes. Generally it was found desirable to reduce this to 5 minutes if best results were to be obtained.

The adoption of uniform delivery of concrete by pumpcrete made practical continuous or semi-continuous placing. Subject to the discontinuance of delivery of the concrete to one tremie to serve a second tremie, the flow was continuous and at a rate that gave good movement out of the bottom of the tremie. Actually, it was found that the capacity of the 12-in. tremie pipe was sufficiently greater than the capacity of the 8-in. pumpcrete delivery, so that there was a tendency for the concrete to become segregated as it dropped from the pumpcrete pipe through the hoppers and the long elephant trunk into the tremie. Had the tremie been smaller, the head differential would have been just as effective in delivering the concrete, but because of less area, the flow would more nearly have been that of the concrete as delivered. This was attested by the control for the trial sections on which 10-in. tremie was used. However, even with this condition, the seal was not often lost. Unless the aggregate mix was carefully controlled, there was a possibility of segregation tending toward the setting up of particle interference producing high internal friction of placing.

Experience on this dock, combined with experience at other places led to the belief that the rate of pour could well be increased in order that there might be a continuous flow of the concrete, eliminating frictional resistance that caused plugging of tremie pipes and segregation of the concrete in the mass. As the operating personnel became experienced with the handling of tremies, it was noticeable that the concrete was more uniformly workable, and that the tremies were maintained at the desired depth without delays. Once started, the tremie pipe was raised a little at a time as the pour continued. Unfortunately, in none of the



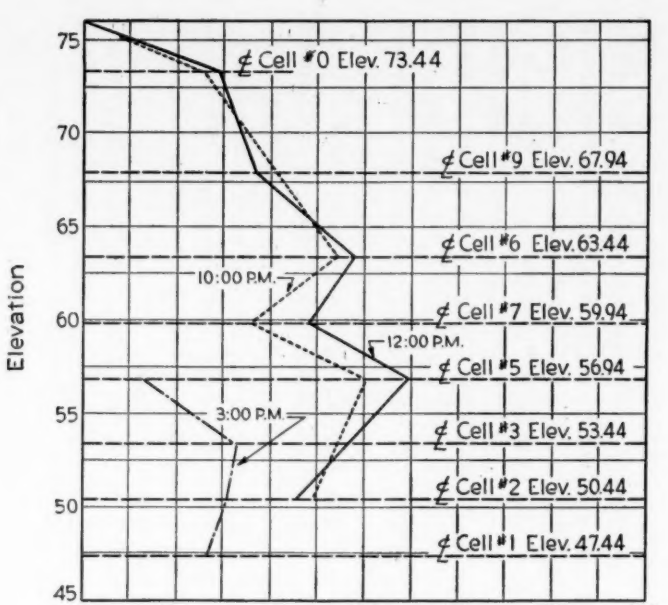


Fig. 5 (top, left)—Pressure distribution-time curve—floor truss 45

Fig. 6 (bottom, left)—Pressure distribution-time curve—floor truss 45

Fig. 7 (above)—Pressure distribution-time curve—wall truss 80-L

tremie operations was it possible to get continuous flow from beginning to end in the floor section as four pumperete pipes served eight tremie pipes.

FORM PRESSURES

Form pressures, developed while delivering concrete under water through pipe tremies, were measured by pressure cells. So sensitive were these cells that small variations and even reversals were recorded which were not anticipated from the measurement of concrete head or flow. Subsequent analysis of the effect of the speed of placing delays, change of tremie pipes, flows from pipes at a distance from the cell, together with the hardening of the concrete justified most of these unexplained readings.

From these tests it appeared that the assumptions made by the form designers as to the intensity and time distribution of pressures had been higher than was actually the case. Not only were the pressures on the forms less than anticipated, but the pressures were reduced materially after the bottom of the tremie was raised above the elevation of the pressure cell. Pressures proved to be a function of flow of concrete as well as of head. Kinetic pressures, experienced at time of flow, were higher than were static pressures when flow had been discontinued.

The form pressure summaries are given in Fig. 5, 6 and 7 which show the time pressure curves for the two groups of cells in the floor pour, and for the group of cells in the wall pour.

Fig. 8 shows the method of installing the cells in the channels, and the installation of the channels in the form. The support of the channel in the truss made possible its ready release after the concrete was in place.

The pressure cell

The pressure cell is about $\frac{7}{8}$-in. thick and 12-in. in diameter. Its component parts are a top plate, base plate and cover plate. Pressure on the top plate (flexible by virtue of a narrow slit milled in its periphery) is transmitted through light transformer oil sealed between it and the base plate to diaphragm formed by boring the base plate. The thickness of the diaphragm varies—depending on the pressure for which cell is designed. An electrical resistance strain gauge is cemented on the center of this diaphragm. A dummy gauge, which is an exact duplicate of the gauge of the diaphragm, is cemented on a metal plate which is tacked to the side wall so that it will not be subjected to any stresses resulting from external loading. The cover plate is fastened to the base plate by bolts and a gasket seals the chamber containing the strain gauges. A three-wire electric cable is brought into the chamber through the side and adequate provision is made to insure that no leakage can occur at this connection.

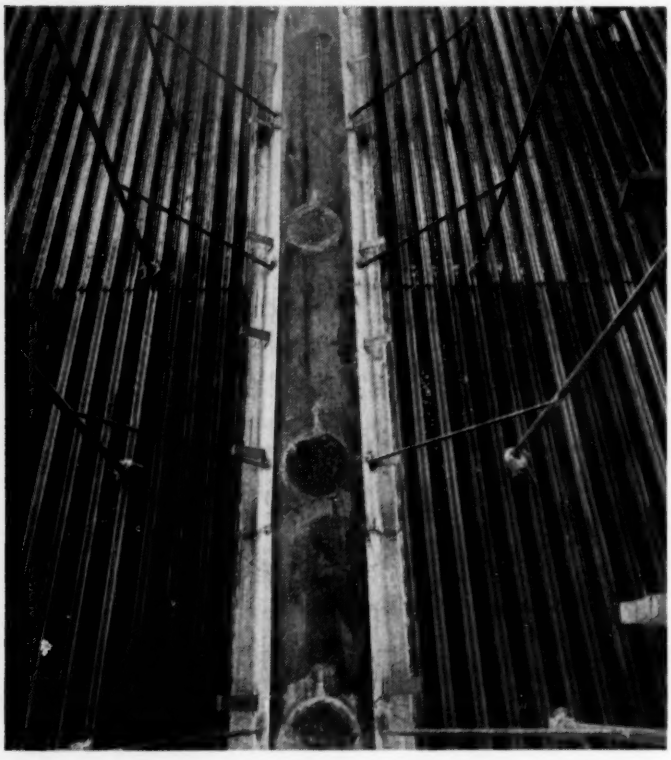


Fig. 8—Cells installed in channel and channel so held in place in form as to be removed readily after concrete was in place

External pressure on the cell exerts a pressure in the liquid contained in the ⅛-in. space in the top. The liquid pressure causes a deflection of the diaphragm, the strain in which is measured by a change in resistance of the electric wire resistance strain gauge.

The active and dummy gauges act as two arms of a Wheatstone Bridge and the recording bridge* as the other two arms. If the recording bridge is initially balanced so that no current flows through the circuit, any unbalance due to a change in resistance of the active strain gauge is reflected by an unbalance of the bridge resulting in the deflection of the galvanometer. The necessary change in resistance in the recording box to rebalance the circuit is a measure of the deflection of the diaphragm and of the external pressure applied to the cell.

The dummy gauge also serves the purpose of compensating for temperature. Since it is identical with the active strain gauge and is placed adjacent to it, any change in temperature that occurs will affect both gauges in the same manner, and will be balanced out in the electrical bridge circuit.

*Described Baldwin Southwark Bulletin 155.

Ten of these cells were manufactured by the U. S. Waterways Experiment Station, eight for use in floor truss 45, and two replacements for cells found unsatisfactory when recalibrated for use in wall truss 80-L.

The cells were installed in the forms prior to setting the truss and leads were rolled on spools to protect them during the lowering of the truss.

The Wheatstone Bridge, a Baldwin Southwark S. R. 4, was found to be sensitive and at the same time appeared to be rugged. If the circuit or the cell were not correctly connected, or if there were water in the cable, the galvanometer picked up the difficulty immediately.

The electrical connections from the cells to the Wheatstone Bridge, located in a building on shore, were made with a three-conductor type S cable. This was extended by the splicing of a four-conductor, ten-gauge cable in the case of the floor truss readings. Difficulty was experienced in obtaining splices that were water-tight, but this difficulty was overcome prior to taking the readings.

The concrete, mixed in dual drum mixers, was delivered to the hoppers above the tremie by pipes extending from the pumperete equipment. The practice followed on the floor truss pours was to commence concrete operations in No. 1 tremie and build up the dome of concrete at this point until the surface of the concrete slope reached a point two feet above the bottom of tremie No. 2, after which concrete was placed in both No. 1 and No. 2 tremies until the surface of the flow reached two feet above the bottom of No. 3 tremie. In this manner, sufficient concrete surrounded each tremie bottom, to allow pumping out the tremie and the starting of concrete placing without loss of seal.

When more than four tremies were delivering concrete, it became necessary to service more than one tremie from a single pump line. This intermittent placing and stopping caused hardening of the concrete,which, at times, resulted in clogging of the tremies. When it was possible to place the concrete more rapidly, the flow characteristics were better; there were no delays from clogged tremie pipes and the tremies could be kept at greater depth thereby resulting in flatter surface slopes.

To evaluate the relationship between pressures recorded on the cells and the head of concrete, careful soundings were progressively made at various points as the concrete was placed. These data were plotted and analyzed to determine the effect of flow and head of concrete on form pressures. Each record was checked against time and tide elevation. Fig. 9 shows the location of the stations at which soundings were taken. After a careful weighing of samples of water taken from the dock, a weight of 63.4 lb. per cu. ft. was accepted for computing hydraulic pressure.

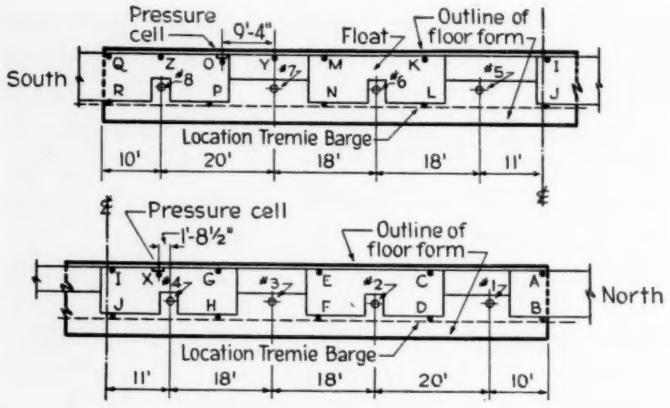


Fig. 9 — Location of soundings and pressure cells—floor truss 45

Each sounding and each Wheatstone Bridge micrometer reading was recorded with the elevation of the tide at the time of taking the reading. Throughout the calculation the hydraulic pressures were obtained by multiplying the difference of elevation between the level of the water and the center line of the cell by 63.4.

In reporting the results of these tests, the fully hydraulic pressure has been subtracted from the indicated pressure to determine the concrete pressure. This procedure is correct as long as the concrete is still plastic and may be correct even after the concrete has hardened as, actually the only pressure in which the form designer is interested is the excess of concrete pressure over hydraulic pressure. On the other hand, if the forms are sufficiently rigid so that cell pressure will not be affected by tidal fluctuations, the pressure of the already hardened concrete may not be truly reflected by subtracting the full hydraulic head. It seemed desirable to follow this method to obtain the net pressure on the forms.

The relationship between the rate of hardening of the concrete and the characteristic of the flow from the tremies into and through concrete already placed has a marked effect on the pressure cell readings. The velocity through the tremies and the remoldability as discharged effects this hardening. Delays or shut-downs allow the concrete to stiffen and cause a change in flow characteristics. This relationship is shown in Fig. 10 and 11, which plot the elevation of the concrete with respect to time and the elevation of the bottom of the tremie for each elevation of the concrete. Superimposed on these curves is a pressure curve for each cell on which the pressure is plotted against time.

Analysis of the effect of the rate of pour on pressures is enlightening, the rate being given by the slope of the concrete placing curve. Attention is called to the effect of distance of the tremie and location of the bottom of the tremie with respect to the pressure cells. Of particular

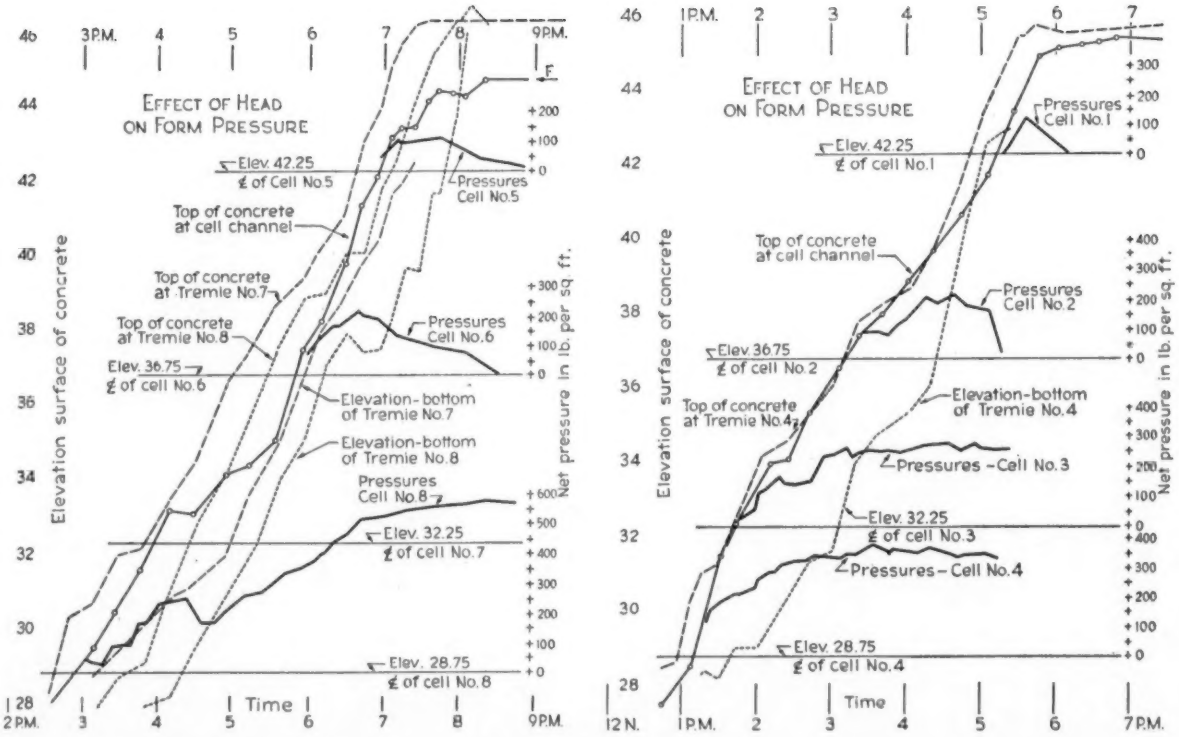


Fig. 10 and 11—Graphs to show effect of head on form pressure plot elevation of concrete with respect to time and the elevation of bottom of tremie for each elevation of the concrete. Superimposed is a pressure curve for each pressure cell, plotted against time

interest is the close time relationship between peaks of pressure and marked changes in the slope of the pressure curve and the time at which the bottom of the tremie was raised above the elevation of the top of the cell, even though the tremies were not less than six feet from the cell.

As long as the bottom of the tremie remained below the cell, there was a constantly changing pressure due not only to the building up of head pressure of concrete on the cell, but also to the effect of the flow of the concrete from the discharge of the tremie outward and upward past the cell. As the bottom of the tremie approaches the elevations of the cell, this pressure may increase and then diminish soon after the bottom of the tremie reaches the top of the cell. This supports the contention that the use of the foot valve or gate on the bottom of the tremie controls the flow and eliminates retempering of the concrete.

The data, obtained during the investigations used in arriving at conclusions, included the micrometer readings taken on the Wheatstone bridge; the change in water level, due to tide variations; the head of concrete on each cell, recorded against time; the calculated hydraulic pressure, and the resultant concrete pressure. The indicated pressures were taken from the calibration curves furnished by the Experiment Station to which were added supplementary pressure scales. In arriving at the scales the micrometer readings, taken after the cells were in place, were evaluated against known hydraulic pressure head. This method was entirely satisfactory as the pressure-micrometer reading relationship was represented by a straight line.

In the case of the readings on wall truss 80-1, that method could not be used as the curves furnished by the Experiment Station, after recalibrating the cells, showed the pressure-micrometer relationship to lie on a curve instead of a straight line. An air zero reading was, therefore, determined prior to lowering the cells and was rechecked by readings taken under water for each cell. This air zero, subtracted from the micrometer reading, resulted in the cumulative micrometer reading used in determining the indicated pressure.

From both of these indicated pressures the computed hydraulic pressure for the measured head of water on the cell was subtracted, and the difference recorded as concrete pressure.

Curves were prepared showing the relationship between time, pressure and head of concrete on the cell. Of particular interest is the effect of delays as compared with pressures developed by uniform continuous placing of the concrete. If the concrete hardens in from 30 to 45 minutes, the form pressures will be lower than if the flow is maintained and the concrete remains plastic for a longer time.

It is of interest to note that the limit of head affecting the increase of pressure is approximately ten feet and that even after a delay, the effect

of head may continue to this height if the concrete has not hardened. In the cells located toward the top of the pours, vibration apparently affected pressures and had an effect on the relationship between head and pressure.

ADVANTAGES OF RAPID CONTINUOUS FLOW

An analysis of the studies on form pressures showed that the tremie location had a marked effect on the recorded pressures. A peak in pressures was obtained at about the time the bottom of the tremie pipe passed the top of the cell, and although in some instances subsequent pressures due to the head of concrete were greater, the effect of the change from the kinetic energy of flowing concrete to the static head of stationary concrete was very apparent. On the other hand, the effect of hardening with elapsed time played a large part, as with the rapid hardening of the concrete, the pressures per foot of concrete head fell off sharply.

The tests showed the effect of the workability of the concrete not only on the form pressure but on the direction of the flow against the form. If the rate of placing was uniform and the flowability of the concrete was constant, a continuing and fairly uniform increase of pressure was recorded. Any delay such as the clogging of a pipe, the change in workability of the concrete, pump breakdown, or any change in the manner of handling the pour immediately affected the pressure. The requirements for design mix for tremie concrete are more exacting than for concrete not placed under water. Only by an appreciation of the differences between the requirements for mass concrete, placed in large batches and solidified by vibration, and concrete that has within itself all of the characteristics of a plastic material that can be extruded into place under pressure, can a successful design of the concrete be developed.

The advantages to be obtained from rapid continuous flow are:

(a) Distribution of the concrete from the tremie by taking advantage of the kinetic energy of the flowing concrete. Flow lines set up remain open and the distribution is more effective.

(b) The reduction of resistance to flow due to stiffening, whether from physical or chemical action within the mass. The elimination of intermittent placing of the concrete assists in maintaining the continuous movement of foreign particles, which assured smooth distribution of the concrete to the forms.

(c) The possibility of reducing the slump and incidentally the cement content of the concrete after the flow has been started, for when once started and maintained, a harsher concrete can be placed than if it were necessary to set up new flow lines following an interruption of the placing.

(d) Control of distribution of the concrete without under water washing or segregation by small adjustments of the submergence of the tremie. This is extremely important, because equal internal pressures may be maintained without the disturbance of concrete already placed. Such methods will result in flatter surface slopes. Economy will result because of the reduction of lost time due to plugged tremies.

The original specification required that operations start at one end tremie and proceed consecutively as each succeeding pipe became surrounded with concrete. It later proved advantageous to start in the middle, working both ways, thus shortening the time by adding two successive tremies and reducing by one half the travel for the entrained silt.

Realizing that irregularities of floor surface would increase the cost and the labor required to prepare the tremie concrete to receive the two ft. finished floor, efforts were made to reduce the difference in elevation of the surface of the concrete. Mechanical items such as tampers and vibrators were tried and change was made in the rate of pour near the top of the form. Although improvement resulted, the importance of getting a smooth floor was not appreciated until the dry dock was unwatered. Lack of water channels and surface irregularities required additional labor and expense to remove deposits of mud. Both labor and money were required to chip off concrete above the designated elevation, and a levelling course was required in those areas that had not been brought to grade. Similar results were reported on other dry docks. Construction methods should be developed and the experiments amplified to produce a more uniform surface.

MODEL STUDIES

Model studies were undertaken to obtain first-hand definite information concerning internal pressure and flow of concrete under water, thereby supplementing measurements taken as concrete was placed in the forms under high heads of water. The studies included an analysis of the characteristics of flow and the effect of location, depth and size of tremie. Placing concrete under water through small pipe tremies, taking soundings and, after the concrete had hardened, cutting sections, together with the use of dry sand and models, developed a new appreciation of flow of concrete.

To meet the requirements for the model use, a mix having the same characteristics of flow, plasticity and re-moldability as the concrete being placed in the prototype was necessary. Preliminary trials with lime and water showed this material to be mobile but too sticky. Lime, sand and water tended to separate under water, but a mixture of cement, sand and water in the same proportions as used in the concrete was extremely satisfactory and was adopted for all of the model studies. In this, 139

lb. of sand and 46 lb. of water were used per sack of cement. The resulting mixture had high slump but was sufficiently homogeneous so that there was no separation while being placed under water. In later tests the proportion of sand to cement was raised to 2:1 but the 5½ gal. of water per sack used in the earlier tests remained the same.

Three sizes of tremie were used in making the various tests: first, a tremie 2 ft. 6 in. long and of 2-in. diameter; second, a pipe 3 ft. 6 in. long, and of 2½-in. diameter; third, a pipe 10 ft. long and of 2½-in. diameter.

The first studies were in the use of a tremie 2-ft. 6-in. long of 2-in. diameter which discharged the concrete onto a plate 3 ft. by 3 ft. 9 in. having sides 8 in. high. This concreting was carried on in air. Observations showed that concrete, extruded from the end of the tremie, traveled outward along the bottom of the plate until the friction on the plate, combined with the internal friction of the concrete, caused the concrete to rise near the pipe and to form a dome extending from a crater immediately around the pipe. Continuing flow showed a series of mass movements outward and downward caused, it was believed, by a combination of the pressure at the end of the tremie pipe and the weight of the mass. As the concrete continued to be placed, the dome continued to rise around the pipe and to move outward over the steel plate.

The longer and larger tremie pipe with its higher effective head, increased the weight of the flowing concrete in the pipe and allowed the concrete to be built up higher around the pipe before the negative pressure stopped the flow in the tremie. The characteristics of the flow were similar, irrespective of the height and size of the pipe, as long as the relation existed between the pressure head caused by the height of the concrete in the tremie, and the negative head caused by the weight of the concrete outside of the pipe.

When the tremie was raised, new lines of pressure and new flow lines developed. The continuous flow of the concrete through the tremie maintained somewhat uniform slopes, whereas, delays allowed the hardening of the surface of the concrete, changed the direction of flow and the characteristics of flow under the surface. Deep tremies caused the flatter surface slopes; shallow tremies steep slopes.

A tank 3 by 4 ft., 20 in. deep was next adopted. With a tremie 2½ in. in diameter and 3½ ft. long held 1¼ in. above the bottom of the tank, mortar of the consistency and mix described was poured into the top of the tremie and allowed to distribute itself under one foot of water. Transverse and longitudinal sections were drawn from soundings at various times during this experiment. In each case, the concrete was discontinued long enough for the measurements to be made.

From this experiment it was concluded that the flatter slopes would be obtained by continuous pours, and that the slope of the surface at any

point is a function of the effective head in the pipe after deducting friction, and the mass movement caused by the gravity of the mass at a distance from the pipe.

As long as the concrete below has not hardened, the dome rises at a somewhat uniform rate. The pressures at the bottom of the slope, being more nearly horizontal than those nearer the pipe, cause outward flows until these pressures are resisted by the side walls of the form or internal friction of the concrete. Attention is called to the difference between this observation and the conception developed in the later model studies.

To determine the effect of a foot plate or valve at the bottom of a tremie, a thin steel plate, held by four light rods at a distance of 1¼ in. from the bottom of the pipe was installed on a tremie of 2-in. diameter and having a length of 2 ft. 6 in. To simulate the operation in the field where, with a 12 in. pipe and high head, the tremie is raised every two to four feet, the tremie in the model was raised from time to time so that the bottom of the tremie was from .3 to .5 ft. below the top of the concrete of the pipe. One foot of water was placed over the floor of the tank prior to the commencement of concreting.

Outlines of flow lines and pressure lines were obtained by addition of mineral colors to portions of the concrete. Sections through the blocks reflected the building up of lines of pressure as the tremie was raised. The effect of the sphere of pressure on the shape of the concrete as extruded from the tremie was shown by lines of demarcation between color pours. It will be remembered that the tremie in these cases was raised as concreting proceeded and, therefore, that the head of concrete on the tremie pipe was never over .5 of a foot. The slopes of the layers substantiate the experience gained by later experiments and justify the statement that new concrete forces concrete already placed, which is above the bottom of the tremie pipe, outward and downward as the head of the concrete is built up around the pipe. The flow of concrete in the area affected is a function of the amount and direction of pressure at the mouth of the pipe in combination with the pressure of the mass itself, including not only the weight of the concrete but of the water above the concrete. If the weight of concrete and water at the mouth of the pipe is small in relation to the weight of the concrete in the pipe, the concrete will rise around the pipe until such time as negative head is built up to offset the height of the concrete, when pressure in combination with the weight of the mass will force the concrete further away from the tremie.

These conclusions were substantiated by actual measurements made during the placing of the concrete in the dry dock under a full head of water and concrete. Flow is affected not only by the weight of the mass and the pressure at the lower end of the pipe, but also by the internal friction of the concrete, the surface friction of the bottom of the area being

filled with concrete (whether this be the top of old concrete or newly placed concrete) and by wall pressures.

The placing of concrete in the tank using tremies 10 ft. long and 2½ in. in diameter was repeated in order that more definite information might be obtained concerning the sphere of pressures. The equipment was set up with one foot of water in the tank and the bottom of the tremie without a plate held 1¼ in. above the floor of the tank throughout the pour. Three colors of concrete were used, natural, red and yellow. The first concrete was placed until a dome had been formed. At this point, red concrete was placed in the tremie, which forced the material already in place first upward until the extruding concrete reached the top of the previously placed concrete and then as the sphere of pressures expanded, the sides of the crater of the already placed concrete were forced outward and downward. (Fig. 12) During the placing of the concrete in which yellow had been added to the natural mix, the dome continued to rise and the yellow was finally forced to the top squeezing the red and natural outward and downward.

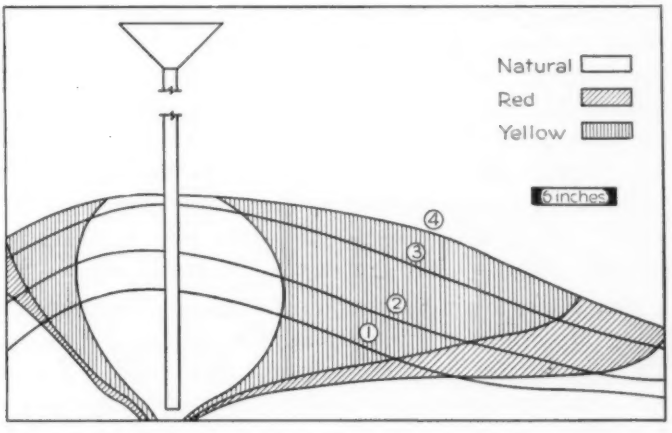


Fig. 12—Section of model pour showing progressive profiles of dome around tremie and final distribution of each color of concrete

A second batch of natural color was then placed and the dome raised while the red, yellow and natural previously placed were pushed outward and downward. It is believed that the cross section gives a true reflection of the lines of force developed while placing concrete in the form. The bottom of the tank acted in the same way as did the foot valves with which the tremies were equipped.

Realizing that variations in the internal friction and particle interference between two mixtures will affect the size of tremie and the flowability of the concrete, it was desired to simulate conditions reported as existing on other projects where coarse, angular aggregates made the flowability of the concrete difficult to obtain. In experiments No. 8 and

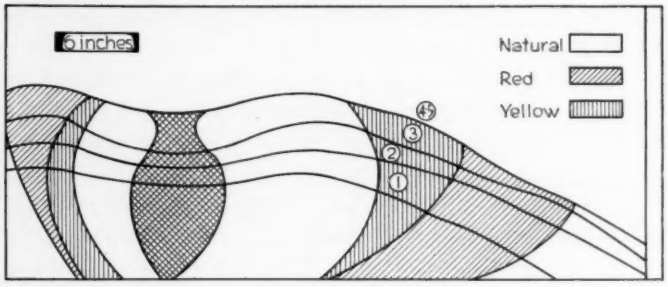


Fig. 13—Section showing final positions of the colors of dry sand forced into place by the lines of pressure and progressive profiles of dome after placing each color

No. 9, dried concrete sand was used as a medium. The tremie used was 2½ in. in diameter and 10 ft. long and the sand was delivered into the same tank that was previously used. The tremie was filled with sand, raised about an inch, the sand allowed to flow and the tremie forced downward. This was repeated until a dome was formed (Fig. 13). Red color was then added to the sand as placed and the raising and lowering of the tremie continued. Subsequently, yellow coloring was added and finally another batch of natural was placed through the tremie. A pumping action forced the material from the tremie into the mass. The surface of the dome moved outward at the sides as it rose in the center. A crater was formed around the pipe and in this crater the red first showed. This red expanded from the crater and forced itself above the natural, forcing the natural to the sides and against the wall. The yellow showed the same forced characteristic as did also the second batch of natural.

Upon the completion of the placing of the material, the pipe was withdrawn and sufficient water added to the tank to saturate the sand. Sections were then cut and sketches and photographs made.

Experience in placing concrete in the dry dock and with model studies, using plastic concrete and dry sand, supports the opinion that when plastic, flowable concrete, or dry sand (made "quick" by sudden vibration) is forced from the bottom of a pipe tremie through the space between the end of the tremie pipe and the horizontal surface of a steel plate foot valve, or between the end of the pipe and the model tank floor, it leaves the orifice in a horizontal direction. But, with the development of the constantly increasing horizontal resistance from the already placed concrete, the pressure is relieved vertically causing a change in direction of the lines of force and the formations of a ball of pressure seen in the sections of the model. As the ball expands it, in turn, pushes in all directions forcing the concrete above it outward and upward and the concrete below it outward and downward until resisted by the bottom and side walls of the tank or form, or by the internal friction of the concrete. Sections cut horizontally and vertically reveal circles or elipses, at times deformed by the external pressure.

The kinetic energy of the concrete at point of discharge must be greater than the static resisting pressure if there is to be movement from the tremie into the form. Unless steel interferes, pressure will be a function of the remoldability of the concrete and the depth of the tremie in the concrete.

The resultant surface formed around the tremie pipe is a dome. If there were no interference, the dome would continue to rise and expand until the negative pressure offsets the available head of concrete.

If now the concrete placed through the tremie is colored and the head on the tremie is raised enough over the resisting pressure again to start flowing, a new ball or sphere, small at first but constantly expanding, will be built up with its bottom tangential to the base plate. As the size of this ball increases the previously formed dome is split open and the sphere pushes the whole dome down and out as it pushes itself upward. Vertical sections taken at this time look like a cabbage head with the outside leaves pushed outward and downward.

In visualizing this action and motion, it is essential that one differentiate between (1) the pushing up of the released material through an area immediately around the pipe followed by a subsequent flow outward and downward over the top of the already placed concrete, and (2) the upward pressure from within the extruding sphere. If the concrete is not plastic, there will not be flow except when subjected to unusual vibration or shock, which will cause the concrete to become "quick" and react to pressure. This has been demonstrated by the model studies using dry sand.

FLOW STUDIES

Conflicting opinions have been expressed with respect to the direction of flow of concrete discharged from the bottom of a tremie pipe when buried from 3 to 5 ft. below the surface of the concrete, and when discharging at depths of 50 ft. to 60 ft. below the surface of the water.

From the preliminary studies made with models it appeared that concrete when delivered from the bottom of a tremie formed a constantly expanding sphere, or bulb, that forced the previously placed concrete radially. Insofar as the tremie was equipped with a plate foot valve, at a distance of one foot below the bottom of the pipe, the pressure from the extruding concrete was upward and outward.

To substantiate these conclusions a full size experiment was undertaken to determine the direction of flow in the slab. The coloring of the concrete by the addition of mineral pigments, the recording of all factors that might affect the interpretation of the results, and the timing of each operation, was planned and carried through in the prototype.

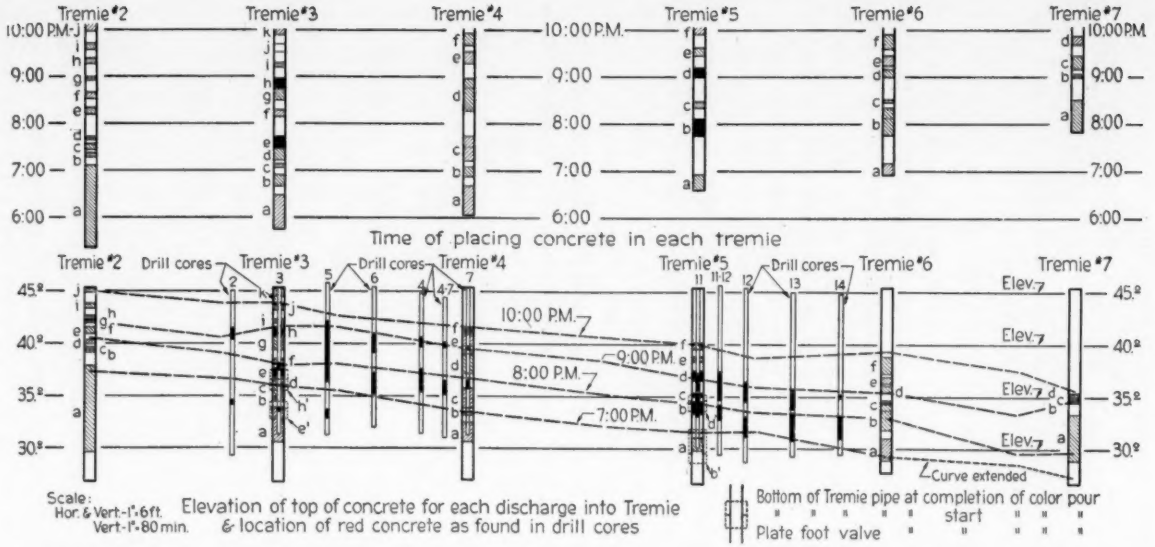


Fig. 14—One of many graphs prepared in the study of tremie placed concrete in dry dock construction which shows the nature of the results. The upper section shows the time of placing concrete in each tremie from 2 to 7. The lower section shows the elevation of each pour increment as well as time profiles and the location of colored concrete found in drill cores

The location of the colored concrete is shown in sections in the drawings. To visualize the time-elevation relationship, enter the top drawing at *e* tremie 3. This concrete was placed at 7:30 p.m. Refer now to *e* and *e'* in the lower drawing. *e* is the top of the concrete, and *e'* the elevation of the foot valve, (approximately elevation 33). Reference to the drawing's distribution of colored concrete shows that this concrete was delivered at this point but was subsequently squeezed beyond tremie 4. In the same manner the second series of colored concretes placed through tremie 3 was marked *h*. These were placed at about 8:50 p.m. The top of the concrete was approximately elevation 42, and the elevation of the foot valve was 36. It will be seen, therefore, that the foot valve was below the top of the concrete at the time the second red concrete was placed.

In a similar manner colored concrete was placed in tremie 5. The flow with respect to time can be successfully followed. This has a definite bearing on the shape of the "lenses" of colored concrete found in the floor as shown by the cores.

No change was made in the time of mix, in quantity of aggregates, or cement per cubic yard. Every effort was made to obtain these data under usual operating conditions. Red iron oxide, to the extent of 5 per cent of the weight of the cement, was placed in the material batches just prior to their discharge into the concrete mixers. Eight such batches were mixed in each of two mixers, one delivering the concrete to No. 3 tremie, and the other to No. 5 tremie. These were followed by eight batches of uncolored concrete mixed in each mixer and delivered to each of the two tremies, and these in turn with six additional batches of red concrete from each mixer. Upon the completion of this run the placing of uncolored concrete was continued until the floor section was completed.

Records obtained included: (a) the time of delivering colored concrete to tremies No. 3 and No. 5; (b) the elevation of the top of the concrete at the beginning and at the end of the placing of each colored pour; (c) time of delivering concrete to companion tremies and the elevation of concrete at the beginning and end of each tremie pour; (d) time and elevation of concrete when placing the second six batches of colored concrete in tremies No. 3 and No. 5; (e) elevation of the surface of the concrete at each sounding point in order that time profiles might be developed by which the direction of surface flows and pressures due to the head of the concrete, might be appreciated. Special consideration was given to the time that the colored concrete reached the tremie and the elevation of the bottom of the tremie pipe and the top of the concrete. The series of soundings taken over the whole surface gave a basis for future study.

After the dry dock was unwatered, cores were taken by drilling into the concrete of the floor. These cores were slightly larger than one inch in diameter. As the drilling proceeded both the drill foreman and the inspector watched the color of the wash water to identify the elevations of the drill at the time that colored water was observed. When the core was withdrawn notation was made of the elevation at the beginning and the end of the drilling, the amount of core recovered, and the amount lost. The core was put into a core box with pieces of wood taking the place of core not recovered. The conditions of the core, the color of the concrete, and the elevation of the colored concrete, were recorded. Later the drill cores were inspected as they were being prepared for final storage and the elevation of colors were checked against the original notes.

The nature of the results of the investigation are shown in Fig. 14. The upper section shows the time of placing concrete in each tremie from No. 2 to No. 7, inclusive. The lower sections show the elevation of each of these increments as well as the time profiles and the location of colored concrete found in the drill cores. The intensity of the flow of concrete from tremie No. 4 between 8 and 9 p.m., was sufficient to divert the

direction of flow of the colored concrete from tremie No. 3. On the other hand, red concrete from tremie No. 3 did travel 20 ft., just prior to 8 p.m. to fill a space in the vicinity of tremie No. 4. Detailed contour maps showed the elevation of the bottom and the top of the colored concrete, and reflect the final slope of the color in the floor. This full size experiment in which colored concrete was used and cores were taken confirmed the observations made during the model study, the form pressure investigation, and the observations of the properties and behavior of plastic concrete under water. It gave reassurance that concrete placed by tremie method is uniform and can be placed without surface washing or segregation. The great uniformity of aggregate distribution, the lack of water pockets or water films around the aggregate combined with the high density of concrete found in all cores removed, insures durability.

CONCLUSIONS

From these studies and observations it is concluded that:

1. The size of the tremie affects the distribution of the concrete and the kinetic energy available for placing, through the control of velocity or discharge, at a predetermined rate of concrete delivery.

2. The distribution of concrete between the tremies will depend on the size of the sphere of influence which, in turn, is affected by the workability of the concrete, the velocity through the tremie, and the depth of embedment of the tremie pipe in the concrete.

3. Tremies used on this project with foot valves set up spheres of pressures which conform to those found in the model as is shown by pressure cell studies which reach the peak soon after the tremie is raised above the elevation of the top of the cell.

4. As the concrete was extruded, the flow of concrete set up lines of pressure which cause the concrete to move upward and outward as the sphere or bulb of pressure expanded. The size and shape of the pressure bulb was affected by the embedment of the bottom of the tremie in the concrete.

5. The internal concrete movements developed in the pour were similar to those developed in the model studies.

6. The concrete was subjected to pressure and movement throughout that portion of the depth of the concrete above the bottom of the tremie. As additional concrete was forced from the tremie, the resultant movement caused flow outward and in places forced the previously placed concrete much as a growing cabbage head forces the older leaves to part at the top and to be flattened toward the ground to give room for the expanding head. When the tremie was raised, the location, force and

direction of the sphere of pressure were changed, which caused an outward movement of the mass acting at a higher elevation.

7. Pressures from tremie pipes equipped with foot valve are transmitted through the concrete in all directions except straight downward.

8. The internal concrete pressure lines developed in the pour were similar to those developed in the model studies. With tremies equipped with foot valve, the pressure cell readings reached the peak soon after the tremie was raised above the bottom of the cell.

9. The form pressures, as measured, were lower than had been assumed.

10. In general the pressures continued to build up to a maximum until the concrete reached a head of approximately 10 ft. above the cell, after which the pressure fell off. Vibration, location of the tremie, elevation of the bottom of the tremie with respect to the cells, and rate of stiffening of the concrete affect the pressure.

11. Maximum pressures at various points in the height of the form do not occur concurrently with maximum pressures at equal heights elsewhere. The slope of the concrete in the floor truss caused maximum pressures to occur at points of variable elevation and at different distances from the center line of the dock.

12. Stiffening of the concrete, whether due to hardening because of lapse of time or to rapid setting action due to heat of the cement or temperature of the concrete, materially affects the flow of the concrete and the transmission of pressure. The flow of the concrete from the tremie toward this cell even though it be through several feet of concrete causes the building up of pressures greater than those caused by static head. The cells are sufficiently sensitive to reflect small changes in pressure

13. Side wall pressures and the presence of reinforcing steel and structural members affect the flow of the concrete from the tremie in the prototype.

14. The sphere of influence is greater and extends over larger areas when the tremie is carried deep than when it is kept at less depth in the concrete. The deeper tremie causes less flow of new concrete at the surface of the old than does the shallower tremie. At the edges of the expanding sphere of pressure there is an internal movement within the already placed concrete.

15. A feature essential to future work is the improvement of the surface levelling by pursuit of recommendations noted and the continuance of the studies on baffling and vibration.

16. The spherical shape taken by the last material placed in the models substantiates the experience with the prototype that the surface

slopes are less steep when the tremie is carried deep than when it is shallow.

17. The movement of tremie concrete is not a surface flow but an internal flow propelled by forces within the mass which cause the newly delivered concrete to replace and force outward concrete previously discharged from the tremie.

18. All investigations are reassuring that concrete placed by this tremie method is uniform, and can be placed without surface washing or segregation. The great uniformity of aggregate distribution, the lack of water pockets or water films around the aggregate, combined with the great density of the concrete, found in all cores taken, insures durability.

Discussion of this paper should be submitted in triplicate to the ACI Secretary before September 1, 1943, for publication in the JOURNAL Supplement concluding Proceedings V. 39.

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Discussion of a paper by P. J. Halloran and K. H. Talbot:

The Properties and Behavior Underwater of Plastic Concrete*

DISCUSSION BY EUGENE E. HALMOS, F. R. HARRIS, A. TAUBER, GEORGE T. GILMAN and the AUTHORS

By EUGENE E. HALMOS†

The tremie method of placing concrete is more than 35 years old but, to the writer's knowledge, was never before subjected to scientific study. The authors are to be congratulated for opening up a new field to investigators of an important method of construction.

Generally, the paper records observations, and gives conclusions based on them, (1) on the movement of concrete in the tremie and in the form, and, (2) on the pressures of the freshly placed concrete upon the sides of the forms. In what follows, the writer will discuss the first of these subjects only.

The flow of concrete is a purely hydraulic problem but is not so treated by the authors. True, scattered over the paper, pressure head, velocity and friction are repeatedly mentioned, but no attempt is made to correlate these basic factors with the observed phenomena except in a rather vague qualitative manner.

It is stated on p. 468, third paragraph:

> The advantage of rapid placing of concrete was apparent when in finishing a floor section it was possible to deliver concrete from two pumpcrete lines to a single tremie. The velocity in this 12 in. pipe served by a single pump is only 30 ft. per minute. When this velocity was doubled, the advantage of the added kinetic energy of flow in carrying the concrete to the forms and around structural steel members was apparent.

Similar statements are made in other parts of the paper. The following quotation from the third paragraph, p. 472, is typical:

> Thus, it became essential that the concrete be placed in final position as quickly as possible by continuity of flow.

*ACI JOURNAL, June 1943; *Proceedings* V. 39, p. 461.
†Engineer of Design Review, Dry Dock Engineers, New York.

Obviously, in the authors' opinion, a high velocity of discharge from the mouth of the tremie pipe is desirable. How much this should be numerically and how to obtain such velocity economically is a matter of considerable theoretical study and practical experimentation in view of the several limitations surrounding the problem. The most important of these is that the volume of concrete to be discharged per unit of time is constant and is determined by the dimensions of the form and by the filling rate specified. Another is the requirement that the rising surface of the concrete be kept as nearly horizontal as practicable; this determines the maximum spacing of the tremie pipes and the actual volume of discharge per pipe. According to the above quotations, the velocity was doubled by doubling the rate of discharge from the tremie. This, of course, is not permissible as it would reduce the specified time of filling the form, unless the operation of every other tremie pipe were discontinued, which, in turn, would result in objectionable, steep slopes of the surface of the concrete in the form.

The velocity of discharge is dependent on the net head acting at the mouth, and on the resistances to outflow produced by the foot valve and by the concrete already placed. The net head is the algebraic sum of the positive and negative hydrostatic pressures at the bottom of the tremie pipe less the frictional losses due to the flow of concrete in the pipe. The hydrostatic pressures can be established from the elevation of the concrete in the tremie pipe, the elevation of the water and that of the concrete outside the pipe. Should the co-efficient of friction for the flow of concrete in steel pipes and its variation with the diameter of the pipe be known, the resistance at the discharge end of the pipe could be evaluated with reasonable accuracy for the conditions prevailing during the test; thus, improvements in the arrangement of the foot valve and in the selection of the proper depth and size of tremie pipe could be made on a rational basis in order to obtain any desired rate of flow.

The writer believes that any further study along the lines initiated by the authors should include a determination of the frictional characteristics of the flow of concrete in pipes, over a reasonable variation of pipe diameters and concrete mixes. Without the knowledge of this all-important factor, a statement such as: "It was believed that a 10-in. tremie would have been preferable to the 12-in. tremie specified and adopted" (p. 467-468) does not seem warranted. The reduction in diameter may have increased the friction to a point where the velocity might have been reduced in spite of the increased head.

The writer thinks that the model tests to show the movement of the concrete in the forms were ingeniously planned and carried out. They give an excellent demonstration of the manner by which the flow takes

place after the concrete leaves the tremie. However, it is believed that the terms "bulb of pressure" and "lines of pressure" are used somewhat loosely. The expression "lines of pressure" is generally understood to denote the loci of equal pressures in a homogeneous mass. In this instance, the lines shown on the diagrams are merely the lines of demarcation between concrete pours of different colors, along which lines the pressure varies from a maximum, at the greatest depth, to zero at the surface. The writer is unable to understand the meaning of "the bulb of pressure," in connection with the model tests.

The emphasis laid by the authors on the importance of continuous supply of concrete to the tremie is of the greatest interest. The plant and arrangement devised to obtain continuity of flow is believed to be novel for placing tremie concrete and deserves great credit; it not only produced heretofore unheard of output but also a concrete of excellent quality.

By F. R. HARRIS*

Captain Halloran and Mr. Talbot are to be congratulated upon the first attempt I have seen to carefully observe and record the behavior of concrete when flowing through tremie pipes into forms placed under water at considerable depth and pressure.

In the introduction the impression is given that the writer conceived the possibility of adopting tremie concrete for the construction of large dry docks without previous application of this method of construction in dry dock work and for the sole and special purpose of applying it to the large Naval dry docks built as a war measure.

Such a conclusion would, however, be in error. As is well known, concrete has been placed under water by the tremie system in various structures for many years past; in fact, in building the first Pearl Harbor dry dock, 1909-13, 8 feet of the bottom of that structure was placed by tremie, the concrete being deposited in rectangular blocks, the forms for which were placed by divers. In the writer's opinion, which he has always adhered to, the failure of this structure in February 1913, could in no wise have been contributed to by the tremie concrete but was brought about by other causes. However, the very fact that a failure of great magnitude occurred, prejudiced many against employing concrete placed by the tremie method in dry docks. The writer, however, was always convinced that if the flow of the concrete were properly controlled and the concrete confined to an area within a reasonable flow radius from the tremie pipe, this method should give satisfactory results.

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In 1927, when acting as the consulting engineer for the Westchester Park Commission on the foundations for the Glen Island Bridge, the opportunity was presented of applying tremie placed concrete within the confines of a small circular cofferdam, and this with great success, so that in the design and construction of the graving dock for the Todd Shipyards Corp., Robins Dry Dock & Repair Co. plant, the outer 220 ft. of this dock was built by placing the floor concrete by tremies. This structure embodied, in many ways in a pioneer and experimental form, most of the features that were later included in the large dry docks referred to by Messrs. Halloran and Talbot. In building the Robins graving dock, core borings were taken of the concrete and when subjected to test they showed uniformly a higher compression and tensile strength than similar concrete placed in the dry.

All of this is mentioned because it is believed that the actual successful application of the use of tremie concrete in the Robins graving dock construction was an important factor in the decision to apply this method to the larger structures for the Navy, this method of placing concrete being not quite so experimental and untried as would be inferred from the paper of Messrs. Halloran and Talbot.

The historical background and development of tremie-placed concrete in dry docks is fairly well covered in the writer's article published in *Civil Engineering*, June 1942, on examination of which it will be definitely seen that the tremie trusses with corrugated steel sides to confine the flow of each tremie pipe was developed to a considerable extent even in the first application of this method in 1928; in fact, the tremie barge and arrangement of tremies, elevating devices and hoppers are well shown in Fig. A from a photograph, made May 28, 1929, which shows clearly that the early application is definitely the basis of the later developments, including even the tremie barge.

As to the method of handling the concrete, while the pumpcrete method was used in three locations for larger docks, other methods have been successfully used. The original method consisted of a concrete tower, distributing by inclined chutes to all six of the tremie hoppers. This tower, mixers and hoppers were placed on a large barge and the coarse and fine aggregate and also cement were brought to it by water transportation. In the first application of this method of the construction of graving docks by the Navy, the concrete was brought by truck to a transverse gantry spanning the entire structure and then distributed to tremie hoppers by belt conveyors. This method was entirely satisfactory and successful. In other locations other methods have been used, all entirely satisfactory. While the pumpcrete method has its advantages, it does, however, involve a very high first cost which, while

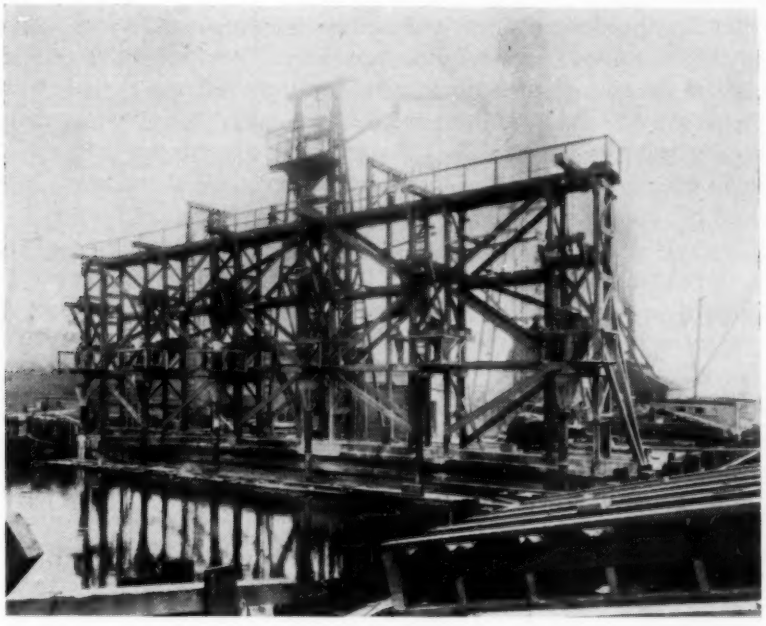

Fig. A

possible of absorption where a large yardage is involved, might readily be prohibitive on a smaller structure.

Referring to the discussion as to the size of tremie pipe, the specifications prohibited the use of a tremie pipe of less than 12-in. diameter. It must be remembered that the specifications were prepared to control not one dry dock project but several of them in widely separated locations, each in the hands of contractors whose organizations, selected plants and many other details differed greatly. In fact, the first of these structures built by this method used a coarse aggregate of broken stone and after experimenting with tremie sizes, used an 18-in. diameter tremie and the contractors even desired to use a larger diameter than this. This project was completed without serious difficulty and tests of the concrete by core borings ran up as high as 9,000 psi, the concrete in general exceeding the estimated weight by several pounds per cubic foot, showing its density.

Another contractor on a similar project which was also in the Pacific area, proposed to use a 10-in. tremie but was not permitted to go below 12-in.

It is noted that the paper states that the hardening was faster than had been expected and that it had been assumed that the concrete would

remain fully liquid for as much as 5 hours. There is some misunderstanding in this statement because the engineers did not expect such results. The initial set of undisturbed concrete was expected in a period of about one hour and the final set in about 5 hours. The 5 hours was more or less an abritrary assumption for the purpose of form design. In the form design it was essential that these underwater forms resist static and kinetic pressure on them as failure of them would have involved, as it did in cases, serious consequences. Such failures occurred in one or two structures, the concrete breaking through the form and overrunning the adjacent area. An order was given to remove this concrete but as some 400 cu. yd. were involved, it was found impracticable to do this without serious delay and excessive expense. Instances of this sort well justify conservative assumptions for form design. The recorded concrete pressures were all very much lower than similarly observed pressures on another structure where they ran nearly three times as high as the maximum given in this paper.

By A. TAUBER*

1. The authors are to be congratulated for their studies, especially in view of the important role the type of construction described in their paper played in the last few years.

The method of placing concrete under water by means of a tremie pipe, movable in a vertical direction only, was used for the first time in 1906, in the construction of the Detroit River Tunnel. The method which was proved successful, was originated by Olaf Hoff, a partner in the firm holding the contract for building the tunnel. (*Proc.* A.S.C.E., Aug.-Oct., 1911).

Mr. Hoff's assistant, A. Gundersen, introduced the new method in Sweden. The Swedish construction firm "Contractor" used it there for the first time in 1911. The Germans, despite the many failures they had on structures where concrete has been placed under water by other methods, had not used this method until 1930. (F. Trier & Tode, *Bautechnik* 1930, 1931). The tremie method is still known in Europe as the "Swedish" or "Contractor" method. Since 1932, according to the German Building Code, it is the only way by which concrete is permitted to be placed "in deep water when high stresses are involved."

To Rear Admiral F. R. Harris (CEC) USN (Ret.) goes full credit for adopting the tremie method in 1927 for the construction of a drydock, where high shearing and bending stresses were encountered. His studies and his experience gained during the construction of this drydock formed

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the basis of all of the drydock projects of late years chosen to be built by the tremie method, because of difficult soil and water conditions.

2. In a properly designed plastic concrete immediately after mixing, the aggregates are completely covered with cement paste. The water not only fills the voids between the cement particles but is in excess of the voids. This not only causes the cement particles to be in suspension in the water, but also places the aggregates, covered by the cement paste, in suspension. Physically a freshly mixed plastic concrete under water behaves as a heavy liquid of the same density as concrete. The submerged weight of the concrete is the difference of the weight of the concrete measured in the dry and of the weight of the displaced water. The net horizontal pressure of the fresh plastic concrete mix under water against a form is the hydrostatic pressure of the submerged concrete.

After the concrete has been placed and left undisturbed, the solid particles start to settle due to gravitational forces. This settlement of the solids causes not only consolidation of the cement paste but also arching of the aggregates. The consolidation and arching of concrete was treated by T. C. Powers, ACI JOURNAL, June 1939, "The Bleeding of Portland Cement Paste Mortar and Concrete."

Parallel with this physical process runs the chemical process of setting. Both processes alter the liquid character of the concrete in place, thus reducing the horizontal pressure against the forms; the arching by mobilizing frictional resistance and the setting by newly introduced shearing resistance.

3. Many attempts were made to measure the pressure of the concrete against the forms. Different devices were tried, but the results have not been consistent. Some of them, similar to the results published by the authors, indicated pressures lower than the hydrostatic pressure of the submerged concrete. On one drydock project extensometers were attached to the tie-rods of the forms. At a 32 foot depth the readings indicated a pressure of about 2,000 lb. per sq. ft., which showed an increase of pressure corresponding to a much greater active depth than that assumed by the designers. On another drydock project Goldbeck cells were attached to the sheeting. At a depth of 6.75 ft. the indicated pressure was 976 lb. per sq. ft., which was almost the full hydrostatic pressure of the concrete in the dry. The reason for the wide variation of the above results is not clear. The results might have been influenced by the fact that all of the devices originally were not constructed to be used under water and had to be altered for this purpose.

In the design of the forms for most of the drydock projects it was assumed that the concrete remains liquid for 5 hours, that the horizontal pressure is equivalent to the hydrostatic pressure of the submerged con-

crete, and that the maximum pressure is reached at a depth corresponding to a 5 hour pour. The designers of the forms were well aware of the fact that the above assumptions did not represent a true pressure distribution; but due to lack of reliable data the design had to cover many unknown factors.

4. The model tests made by the authors show that the "old" concrete is kept moving within a certain "sphere of influence." It is reasonable to assume that within this "sphere of influence" the concrete remains liquid, while outside of it, where it is left undisturbed, it changes its liquid character. The deeper the tremie pipe is submerged into the concrete, the deeper and larger is the sphere of influence. It seems to the writer that, in addition to the rate of pour, the submerged depth of the pipe, which was not specified, (it was down to 12 ft. on one drydock project) has the greatest influence on the distribution of pressures.

5. "The statically balanced head" of the liquid concrete in the tremie pipe is the height of concrete which balances the outside water head. It is a function of the depth of water and weight of concrete only and is independent of the depth of pipe submerged in the concrete. For a 60 foot depth of water weighing 62.5 lb per cu. ft. and a unit weight of concrete of 150 lb. per cu. ft. this head is 25 ft. If liquid concrete is added at a uniform rate in the tremie pipe above the statically balanced height, the concrete will flow out of the pipe at the same time as the concrete will rise in the pipe, until a height is reached at which it will remain constant. The difference of the two heights is the pressure head. Part of the pressure head is used up to overcome viscous resistance in the pipe and in the open channel flow, the loss at the orifice and due to change of direction of the flow. The viscosity of the concrete is a function of its consistency, which in turn is a function of the aggregates and the excess water. The deeper the pipe is submerged in the concrete, the longer the travel and the greater is the loss of pressure head. It is reasonable to assume that for a given submerged depth of tremie pipe, and uniform rate of filling, there will result a certain pressure head which causes the flow. It is regrettable that the authors did not publish any data as to the heights of concrete in the tremie pipe for various rates of filling.

If filling of the tremie pipe is discontinued the concrete will drop below the "statically balanced height," but will then rise and finally come to rest at the balanced height. The authors report that "even short delays caused a stoppage of the flow . ." (p. 473) is in agreement with similar reports by others.

The arching of the aggregates, as mentioned previously, sets in immediately. It can be proven, that a few feet deep "arched" concrete in the tremie pipe is sufficient to stop the flow even if liquid concrete is placed

up to the top of the tremie pipe. The added weight is carried by the arched concrete similar to the weight carried by granular materials in bins. If arching is far developed it may be broken only either by lifting the tremie pipe out of the concrete, by vibration, or by some other mechanical means which, when applied to the arched concrete, changes its character back to a liquid.

By GEORGE T. GILMAN*

The authors are entitled to the thanks of the engineering profession for the worthy contribution which they have made to the rather limited published data based on experience in the placing of large masses of tremie concrete in underwater forms.

To the writer's knowledge, very few structures of reinforced concrete had been built by tremie methods before the Navy's recent program of graving dock construction got under way. Many bridge pier foundations have been constructed at least partly by underwater methods, either by bucket or tremie pipe but in most cases these have been of concrete without reinforcement. In 1931, the writer was Resident Engineer on the construction of the foundations for a large building in Pittsburgh. Because of excessive leakage of ground water into the open cofferdam trench, the water was allowed to rise until it reached its static level and the lower part of a section of reinforced basement wall was then satisfactorily placed by tremie. No doubt there are a number of such isolated instances which have not been recorded in technical journals.

The authors were afforded an unusually favorable opportunity for pioneering work because of the daily repetition of the pouring, underwater, of these large, reinforced concrete monoliths. It is believed that the adoption of the pumpcrete method for delivery of concrete to the tremies was also a unique feature as applied to tremie work. This was probably the one distinct difference between the tremie concreting methods which were employed for the construction of these graving docks and those formerly used.

Apparently 12-in. pipes were satisfactory for the work described in this paper, but it appears to the writer that *for the usual grading of aggregates* and using concrete with a slump of 7-in., as cited by the authors, the necessity, or even the desirability of using pipes as large as 12 in. is questionable. Tremies of 8-in. diameter have delivered more than 100 cu. yd. per hour on bridge piers where this was as much as they could be supplied. Pipes of a size which would seem to be about the upper extreme were recently employed on a dry dock project where the contractor convinced the engineers that 18-in. pipes were more satisfactory

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for their particular job conditions. The only statement made by the authors bearing on this point is as follows:

> Had the tremie been smaller, the head differential would have been just as effective in delivering the concrete, but because of less area, the flow would more nearly have been that of the concrete as delivered. This was attested by the control for the trial sections on which 10-in. tremie was used.

There are arguments in favor of the use of a smaller tremie pipe, but there are no specific data in the paper indicating clearly whether the authors think that this would have been desirable on this project.

Regarding the consistency of the mix, earlier tremie jobs frequently made use of a stiffer concrete mix than was considered to be necessary and satisfactory (in the sense that it yielded adequate strength, etc.) for this tremie dock construction.

Concerning the grading of the mix, the authors make the following statement:

> Gravel was delivered in one size only and a grading was selected that would not be objectionable even with some segregation. In other words, the grading was on the coarse, rather than on the fine side.

This would seem to require explanation since segregation is always objectionable and it would appear that the coarser the grading, the more objectionable it would be. A smooth plastic concrete is as necessary for tremie work as for concrete placed in the dry.

Reference is made by the authors to the kinetic energy of the concrete after it has been extruded from the bottoms of the tremie pipes. Except for the possible sloughing of a mass of concrete which has built up to a mound and then flows rapidly on some cleavage plane, it would not seem that the kinetic energy of concrete moving largely horizontally but with some upward component, would be an appreciable factor in form pressures. The velocity of the concrete passing down the tremies is low, probably not more than 1 to 1.5 ft. per sec. under average operating conditions and after the concrete has mushroomed out a few feet from the bottom of the tremie, the velocity is but a small fraction of the velocity in the pipe. The thought occurs that the behavior of the pressure cells used for determining form pressures (to which the kinetic energy of the moving concrete has been assigned as an explanation) is more likely explained by the sensitivity of the cell itself. The cell may hold one reading for a period and then "jump" to a higher reading under the influence of the vibration effect. This undoubtedly is present to some degree due to the dropping of concrete into the tremie pipe from the hoppers above.

The following paragraph regarding design vs. actual form pressures is quoted from the paper:

> Observations of concrete as placed in the forms showed that the rate of hardening was faster than had been expected. It had been assumed that the concrete remained

fully liquid for as much as five hours and the concrete adjacent to the tremie pipes remained fluid for a longer period than that adjacent to the walls. It soon developed that instead of the concrete remaining plastic for five hours, the surface of undisturbed concrete lost plasticity within 45 minutes.

There seems to be a confusion here between "design assumptions" and what might be spoken of as "expectations." It is commonly known that concrete will "stiffen up" in from 45 minutes to an hour or thereabouts, and, obviously, if conditions are such that the concrete is not retempered, the concrete will not remain "fluid" for five hours, but only for a small fraction thereof. Safety was of the essence in the design of the underwater forms. Great delay might have ensued had there been a major failure because it would not only have involved cleaning out the concrete from adjoining sections into which it might flow, a stupendous task in itself, but also many other forms, some already in place, others at the site ready to be placed, and still others in assembly yards in the course of fabrication, would have required strengthening, additional members, etc. with consequent delays which simply could not be allowed. Also, the economies that could be effected by the use of lower form pressures are not in proportion to the pressures. The requirements of stiffness for handling and lowering these tremendous form sections were in themselves such that the total weight would not have been reduced greatly by lower assumptions as to form pressures. Also, the dynamics of concrete flow in underwater forms are not fully known and such phenomena as the sloughing of a large mass of concrete against a form panel was considered likely upon occasion. All these things and many others entered into the selection of a design pressure diagram for the forms which is reasonably conservative.

Concerning form pressures of concrete poured in the dry, reference is made to an article by H. P. Maxton, General Cost Engineer, Tennessee Valley Authority, (*Construction Methods*, June, 1940). A graph of form pressures resulting from extensive studies on TVA projects is given therein and the following tabulation is taken from that graph for concrete poured at temperatures of 70 and 40 F. respectively.

Pouring Rate in Vertical Ft. per Hr.	Depth of Concrete When Max. Pressure Occurs, Ft.		Time During Which Max. Pressure Develops, Hr.		Max. Pressure Lb. per Sq. Ft.	
Degrees F.	70	40	70	40	70	40
2	4.2	6.0	2.1	3.0	370	530
3	4.8	8.0	1.6	2.7	480	720
4	5.6	9.8	1.4	2.4	570	900
5	6.0	10.6	1.2	2.1	650	1050
6	6.2	11.5	1.0	1.9	710	1180

It is to be noted that the pressures given are for "concrete of fluid pressure equal to 140 lb. per sq. ft."

According to these data, the maximum pressure does not increase in proportion to the pouring rate and the absolute maximum pressure is independent of the pouring rate. For concrete placed at a temperature of 70 F., the absolute maximum pressure would be about 800 lb. per sq. ft. at a depth of a little more than 6 ft., but for a 40 F. placing temperature it appears possible that the maximum might run well up toward 2000 lb. per sq. ft. at a depth of 15 to 20 ft.

It should be remembered that the form pressures used for design were chosen with the knowledge that placing rates might at times considerably exceed those recommended and that the effect of possible low pouring temperatures should be guarded against.

Concerning placing temperatures and temperatures generated in the mass by the heat of hydration, the authors do not speak at length, referring only to their observation that "the temperature of the cement and the temperature of the concrete proved vitally important factors in the workability of the concrete." Very little has been published on temperatures in tremie concrete. H. A. Moody gave interesting data (*Engineering News-Record*, Aug. 20, 1936) on the setting temperatures in a block of tremie concrete placed at 82 F. in water at freezing temperture. Four calibrated resistance coils were placed from one to eight feet below the surface of a mass about 9 ft. thick. All except the upper cell showed setting temperatures from 150 to 160 F. from 12 to 18 hours after placing. The upper cell, one foot felow the surface showed a maximum of 130 F. and nine days later the temperature of the concrete at this point, so near the surface where the water temperature was 32 F. was a little above 60 F. The deeper cells showed temperatures of 120 to 130 F. at this time.

It therefore appears probable that the setting rate of mass tremie concrete will be affected much more by the temperature of the concrete as placed than by the temperature of the water surrounding the mass.

From the authors' Fig. 10 and 11, it appears that recorded maximum pressures varied from 115 to 580 lb. per sq. ft., that they occurred from 15 minutes (at top cells) to 5 hours and 30 minutes after the cells were covered and at concrete heads from 2 ft. to 13.5 ft. It is apparent that under the job conditions as to placing rates and probable temperatures, only re-tempering could account for the higher figures cited.

An extremely interesting point brought out by the authors is their recommendation that further studies be conducted to develop an effective arrangement of foot plate and baffles to control the direction of flow and permit shallow submergence of the tremie pipe. Their observa-

tions in this regard are a valuable contribution to the development of improved methods of pouring tremie concrete.

AUTHORS' CLOSURE (based only on discussion by Mr. Halmos*)

It is easy to understand why Mr. Halmos, in his discussion, has not fully understood some of the principles embodied in the study of plastic concrete under water. The paper was condensed from a large volume of papers, tables and diagrams, so that it might be brought to a reasonable length for publication in the ACI JOURNAL. Thus, there are many phases which have to be excluded that are brought out by many other diagrams.

In his paragraph 3 Mr. Halmos states "the flow of concrete is a purely hydraulic problem but is not so treated by the authors." The authors takes exception to the statement that the flow of concrete is a purely hydraulic problem. By definition, "hydraulic" applies to the science of liquids, as water in motion; "hydraulic" is also applied to a property of certain types of cement which harden under water. Neither of these interpretations applies to the problem of concrete under water. In the first place, tremie concrete does not act as a normal fluid. Otherwise, there would be very little purpose in the paper. "Properties and Behavior Under Water of Plastic Concrete." There are so many attendant factors in the motion of plastic concrete, such as the initial set, the harshness, time of getting into final position, obstacles creating odd and unexpected back pressures, and the direction of flow, all of which make this problem anything but a purely hydraulic problem.

Mr. Halmos has obviously failed to see the correlation of these basic factors, since he states that they are brought out in rather vague qualitative manner. Fig. 4, p. 471, shows the phenomena observed by the authors during the field tests prior to the preparation of the articles. A study of Fig. 5, 6 and 7, as well as 10 and 11, present something of the time element. It is true that the subject matter could better be presented if there were more diagrams.

In the fourth, fifth and seventh paragraphs, Mr. Halmos mentions the desirability of high velocity of discharge from the mouth of the tremie pipe. As he had concluded, it would be unwise to attempt to treat this subject numerically or to deal with it as a problem of economy on the basis of the practices of one or two docks. Again, there are so many factors of time, type of equipment, characteristics of aggregates, temperatures of water, sizes of tremie pipe, etc., that quantitative conclusions drawn would be of doubtful value. It had been hoped that these tests would be duplicated and repeated at other sites. This was attempted at

*Captain Halloran having been detached for service which made him unavailable for participation in the consideration of discussions by Messrs. Harris, Tauber, and Gilman, the concluding discussion is by Mr. Talbot.

one other location, but was not carried on as extensively as at the first location.

The authors realize that their paper but scratches the surface, and that much additional experimentation will be necessary before quantitative conclusions can be drawn.

A reference is made to the necessity of keeping the top level of the concrete as nearly horizontal as practicable. Mr. Halmos seems to have failed to read in the paper the relationship between horizontal surface and depth of tremies. This is an important point, as is shown by conclusion 2 on p. 490.

Mr. Halmos contends that the terms "bulb of pressure" and "lines of pressure" are used somewhat loosely. If they are loosely used in this connection, then they are also loosely used as frequently applied to the studies of soil pressures. The authors do not agree with Mr. Halmos that the lines of the diagram are merely lines of demarcation between concrete pours of different color. They are actually lines of pressure. This subject is brought out under the heading of Flow Studies, p. 487, and also appears in conclusion 17, p. 492.

Mr. Halmos seems to think that it was unwarranted to state that a 10-in. would have been preferable to a 12-in. tremie. Size of the tremie pipe is a factor related to the characteristics of the mix, and when plastic concrete has the good characteristics of workability, or perhaps if we could say flowability, the smaller tremie pipe gives the better control. This not just a belief, but was actually proven by tests wherein a 10-in. pipe showed less fluctuation in the levels of the concrete in the pipe and gave superior results in the finished product. Naturally, this point could not be too greatly stressed, in that it might not apply at locations where the aggregate was less workable than at the point where these tests were conducted.

FURTHER DISCUSSION BY MR. TALBOT

The discussions of Admiral Harris, Mr. Halmos, Mr. Tauber and Mr. Gilman are greatly appreciated by the authors of this paper. Each has made valuable additions. Capt. Halloran's reply to the discussion by Mr. Halmos summarizes the difficulties that faced the authors and the Institute in reducing the mass of data to the length of the paper. It seemed better to cover the whole field of the investigation, even at the risk of having the conclusions appear without proper statistical backup, than to reduce the scope of the paper.

It is apparent from the discussion that there are phases of the investigation that have been misunderstood. This investigation was instituted to obtain information that would be valuable in the control of construc-

tion and could be used on future designs, and was expanded to check impressions gained while placing concrete day and night under sixty feet of water. As the problem before us at all times was production under control, it is not surprising that the size of the tremie, the direction of the flow, the speed of flow and the characteristics of concrete movements became of great daily importance. Tests had been made using tremies smaller than 12-in. in diameter. Concrete was placed on the same project in structures not described in this paper using 6-in. tremies with high rate of delivery.

Ours was the job of getting hold of the underlying flow pattern that gave the best results in the finished drydocks. To do this it was necessary that we know something about comparative form pressures and the flow paths of concrete to the forms. Rate of rise was studied and co-ordinated with the problem of resultant form pressures. The effect of the elevation of the bottom of the tremie on the form pressures was noted, and the investigation extended to determine the reason for the phenomena. Soundings and measurements were taken under all conditions of operation to insure a background of data from which conclusions could be drawn. The form pressures assumed by the designers were used as a standard of comparison. The references in the paper are not a criticism of conservative practice.

Apparently the sphere of pressure referred to as having been developed by model studies has been misunderstood. As explained in the paper, the model studies were developed over a considerable period as a check on opinions or observations found during construction. It has been suggested that the section shown in Fig. 12 reflects only the location of the colored concrete rather than the lines of pressure or the paths of movement. Perhaps further reference to Fig. 12 is justified to clear up any misunderstanding.

Assume if you will a fully elastic balloon, small in size, resting upon the ground. It has the ability to change shape by stretching as much as is necessary. On the outside it is subjected to water pressure. The inside of the balloon is being filled with a material having a weight of 150 lb. per cu. ft. The material is delivered at the bottom of the balloon through a tremie that passes through the top of the balloon. As the filling begins, material under pressure is forced into the interior. At first the filling spreads in the bottom of the balloon horizontally, but soon the balloon begins to grow in height as well as in girth. Lines of force and direction of flow are set up that cause the surface to be higher in the middle than at the sides. When the balloon touches the side form, its shape is changed until it conforms to the form. The profile of the surface may be similar to Line 1 in Fig. 12. If now the color of the filler is changed and the filling is continued, the general shape of the dome may be as shown in

2. Again change the color of the filler and continue until the dome is as shown in 3. Finish placing with the original color of the filler.

As the dome has expanded, the filler delivered at the bottom has assumed its position either by moving up through the center and then flowing outward to the sides or by forcing material already in place upward, sideward and downward. If, then, a section is cut through the balloon, the direction of flow can be determined by the location of the colors. The first uncolored material which at one time formed a dome under Line No. 1, is now found in the bottom, but squeezed to the sides and actually raised along the side walls above its original location. The next colored filler has been pushed down on top of the first material and squeezed into a thin layer between the wall and the later material. The last material placed has forced itself into the middle of the mass and has become the activating medium for the movement of the already placed filler.

The concrete served as both the balloon and filler, no balloon having been used in the model. The pressure of the water on the surface of the concrete while being placed set up a condition similar to that of the ever-expanding balloon. Forces and pressures found in the pressure cells studies justified the contention that tremie concrete as delivered on this project was extruded into place under a combination of pressures that moved the concrete horizontally as well as vertically, filling in spaces on the underside of the forms which could not have been filled had the flow been from the top down the slopes.

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Prestressed Concrete, Design Principles and Reinforcing Units*

By HERMAN SCHORER†
Member American Concrete Institute

SYNOPSIS

This paper discusses the steel stress limitation in ordinary reinforced concrete design and shows how working stresses may be increased by the use of monolithic, prestressed concrete construction. It gives a description of self-contained prestressed reinforcing units and points out the advantageous combination of high yield point steel wire and high early strength concrete. A derivation of the elementary design principles is illustrated by examples. The theoretical conclusions are compared with some observations on prestressed prisms and test beams.

1—INTRODUCTION

The present emergency has revived the discussion of permissible working stresses in reinforced concrete design. To conserve steel, higher steel stresses have been proposed by Bertin,(1) and others. The "National Emergency Specifications for Design of Concrete Buildings," dated Nov. 10, 1942, issued by the War Production Board, include a mandatory provision for steel stresses from 20,000 to 24,000 psi. By stepping up the steel stress and lowering the concrete compressive stress, some steel economies can undoubtedly be effected. They are partly offset by the increased dead load of the structure, heavier foundations and a reduction of usable space. At the same time the problem of cracks becomes more serious, depending on the type, purpose and exposure of the structure.

2—NOTATION

A = area of transformed section.
A_c = area of net concrete section.
A_s = steel area.
a = distance between c.g.c. and n.a.
α = concrete age at loading.
$c.g.c.$ = center of gravity of concrete area.

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(1) See references at end of paper.

$c.g.s.$ = center of gravity of steel area.
$c.g.t.$ = center of gravity of transformed section (neutral axis)
c_1, c_2 = extreme fiber distances, from $c.g.c.$
c'_1, c'_2 = extreme fiber distances, from $n.a.$
E_c = modulus of elasticity of concrete
E_s = modulus of elasticity of reinforcing steel
e = eccentricity, with respect to $c.g.c.$
e' = eccentricity, with respect to $n.a.$
e_p = relative plastic flow deformation.
e_s = relative shrinkage, or swelling of concrete.
F = direct axial load.
f_c = effective concrete stress.
$f_c\alpha$ = effective concrete stress upon release of the prestress force.
f_{cF} = concrete stress change due to axial load, F.
f_{cM} = concrete stress change due to external moment, M.
f_{coP} = effective concrete stress at the $c.g.c.$, due to prestress force, P
f_{csP} = effective concrete stress at the $c.g.s.$, due to prestress force, P.
f_{c1P} = extreme concrete fiber stress, due to prestress force, P.
f_{c2P} = extreme concrete fiber stress, due to prestress force, P.
f'_{c1M} = concrete fiber stress change due to external moment, M.
f'_{c2M} = concrete fiber stress change due to external moment, M.
f'_{csM} = concrete stress change at the $c.g.s.$, due to external moment, M.
f_s = effective steel stress.
$f_s\alpha$ = effective steel stress upon release of the prestress force.
f_{sF} = steel stress change due to axial load, F.
f_{sM} = steel stress change due to live load moment, M.
f_{so} = original steel stress.
f_{sP} = effective steel stress due to prestress force, P.
Δf_s = total steel stress reduction.
Δ_e = steel stress reduction due to elastic deformation.
Δ_p = steel stress loss due to plastic flow.
Δ_s = steel stress loss (or gain) due to shrinkage (or swelling).
h = relative humidity, expressed as a fraction.
I = moment of inertia of transformed section, refererd to $n.a.$
I_c = moment of inertia of net concrete section, referred to $c.g.c.$
I_s = moment of inertia of net concrete section, referred to $c.g.s.$
$k_1 = f_{c1P}/f_{coP}$, stress ratio.
$k_2 = f_{c2P}/f_{coP}$, stress ratio.
$k_s = f_{csP}/f_{coP}$, stress ratio.
L = span of simple beam, also free column length.
l = length of prism.
Δl = shortening of prism.
M = live load moment.
M_o = dead load moment.
M_P = internal moment, due to prestress force, P.
m = steel stress loss factor due to plastic flow.
$n = E_s/E_c$.
$n.a.$ = neutral axis.
P = effective prestress force.
P_o = original steel prestress force.
P_{cr} = ultimate column load.
p = steel area ratio.
$p_1 = S_1/v$, principal stress ratio.
$p_2 = S_2/v$, principal stress ratio.
$p_x = S_x/v$, stress component ratio.
$p_y = S_y/v$, stress component ratio.
Q = statical moment for shearing stress determination.
S_1, S_2 = principal stresses.
S_x, S_y = normal stress components.
S_c = compressive strength of concrete, psi.
S_f = flexural strength of concrete, psi.
S_t = tensile strength of concrete, psi.
S_{c1} = section modulus of fiber c_1, referred to $c.g.c.$
S_{c2} = section modulus of fiber c_2, referred to $c.g.c.$
$S_{c'1}$ = section modulus of fiber c'_1, referred to $n.a.$
$S_{c'2}$ = section modulus of fiber c'_2, referred to $n.a.$
S'_e = section modulus of fiber e', referred to $n.a.$

$s_1 = S_{c1}/A_c$.
$s_2 = S_{c2}/A_c$.
V = total shearing force.
v = shearing stress component.
v_{max} = maximum shearing stress.
x = plastic flow time period.
x/α = relative age of concrete member subjected to a sustained stress.
y = relative plastic flow deformation per unit stress.
φ = angle, defining location of principal stress plane.

3—CRACKS

The practical limit of all working stresses is essentially determined by the low tensile strength, S_t, of all concretes. The flexural strength, S_f, is more easily determined by test. Based on the investigations of Abrams[2] and others, S_t, and S_f, can be expressed as a function of the compressive strength, S_c,

$$S_t = 5\sqrt{S_c} \quad \ldots\ldots (1)$$

$$S_f = 10\sqrt{S_c} \quad \ldots\ldots (2)$$

Within the range of usual working stresses, the instantaneous modulus of elasticity of concrete, E_c, is also a relation of the compressive strength, as shown by Walker,[3] Gruenwald[4] and Graf.[5]

Based on the nomograph given by Walker, the secant modulus of elasticity, E_c, for the stresses given by equations (1) and (2), can be expressed as follows,

$$E_c = 80{,}000\sqrt{S_c} \text{ (direct tension)} \quad \ldots\ldots (3a)$$

and

$$E_c = 72{,}500\sqrt{S_c} \text{ (flexure)} \quad \ldots\ldots (3b)$$

Equation (3a) closely coincides with the average value suggested by Graf, assuming that the prism strength is 85 per cent of the cube strength. Individual variations of about 20–30 per cent must be expected in all these expressions.

If, E_s, designates the modulus of elasticity of the reinforcing steel, f_s, the steel stress and, f_c, the concrete stress, the identical extension of steel and concrete is stated by the relation,

$$f_s = \frac{E_s}{E_c} f_c = n f_c \quad \ldots\ldots (4)$$

With E_s = 29,000,000 psi, and E_c from equations (3a) and (3b),

$$n = \frac{362}{\sqrt{S_c}} \text{ (direct tension)} \quad \ldots\ldots (5a)$$

and

$$n = \frac{400}{\sqrt{S_c}} \text{ (flexure)} \quad \ldots\ldots (5b)$$

Cracks develop when the concrete stress exceeds the approximate limits given by equations (1), (2), (4) and (5) respectively. From equa-

tion (4), the corresponding steel stresses are,

$$f_s = 362 \times 5 = 1810 \text{ psi} \quad \text{(direct tension)} \ldots\ldots\ldots\ldots\ldots\ldots (6)$$

$$f_s = 400 \times 10 = 4000 \text{ psi} \quad \text{(flexural tension)} \ldots\ldots\ldots\ldots\ldots\ldots (7)$$

It appears therefore that the recent improvements in the compressive strength of concrete are largely offset by a reduced value of, n, so that practically all concretes crack at the same low steel stress limits. In addition, the reinforcing bars resist the volume reduction due to drying shrinkage, which action subjects the concrete to internal tensile stresses. So-called "shrinkage reinforcement" can therefore mainly change the width and distribution of cracks.

As an unavoidable compromise, the design must be based on a cracked tension zone, assuming that the monolithic action between steel and concrete still exists. This procedure is justified by the experience that wide cracks can be avoided if the steel stress and the diameter of the reinforcing bars are kept below established limits.

It is evident that much further progress cannot be expected as long as the working stresses of steel and concrete are controlled by other considerations than the best physical properties of these materials.

4—PRESTRESSED CONCRETE

Prestressed concrete construction attempts to eliminate concrete tensile stresses under normal load conditions. The designer then is in a better position to utilize the higher compressive strength of modern concrete and the excellent tensile properties of cold drawn steel wire, such as spring wire, plow steel wire, music wire, piano wire, etc.

The first attempts with prestressed reinforcement were probably made by Lund and Koenen, as reported by Mörsch.[6] However, the result of their beam tests was disappointing, due to a low steel prestress. More recently, Freyssinet has achieved remarkable progress with high yield point steel reinforcement, as described by Mautner,[7] Rosov[8] and Billig.[9]

Finsterwalder[10] and Dischinger[10] have suggested methods which abandon the principle of monolithic construction in order to facilitate application to cast in place structures.

Hoyer[11] has shown that steel wires of less than about $\frac{1}{8}''$ dia. are self-anchoring when embedded in concrete under a high initial stress. The reinforcement used by Hoyer for the fabrication of precast beams consists of hot drawn, lead patented steel wire, with a tensile strength of about 340,000 to 380,000 psi and a yield point of 85-90 per cent. The reported steel prestress varies between 170,000 to 250,000 psi.

The monolithic prestress methods, as used in Europe, are mostly limited to precast members. For cast in place structures, the cost of

special equipment becomes prohibitive, due to the requirement of heavy steel forms or temporary end-anchorage structures, designed to receive the large prestress force reactions. The economies gained by a greatly reduced amount of reinforcing steel are thereby lost in advance.

The investigations reported in this paper were made during the development of prestressed reinforcing units.* However, the analyses and results of tests are applicable to reinforced concrete prestressed by other means.

5—PRESTRESSED REINFORCING UNITS

The self-contained, prestressed reinforcing units, as shown in Fig. 1, are intended to overcome evident practical difficulties in monolithic prestressed concrete construction. These units consist of two groups of spring wire, which are wound, or braided, over a center member, in a clockwise and counter-clockwise direction respectively, so as to balance the torsional moments. The individual wires are spaced from each other, and from the center member, by means of stamped spacer discs, at regular intervals.

The center member consists of a cold drawn and heat treated steel rod or tube, which serves as a temporary compression member. It is enclosed within a paper or sheet metal tube to prevent the entrance of mortar and to permit later removal of the rod.

The reinforcing wires are connected to end members, such as shown in Fig. 2(a) and 2(b), designed to transmit the stretching force reaction to the ends of the center rod. For this purpose the wires are thoroughly gripped by means of a ring wedge clamping device.

The prestress force is applied by means of a hydraulic jack attachment, which simultaneously stretches all wires and compresses the center rod. The load is determined from the readings of a pressure gauge, and is maintained by means of a locking device. Subsequent temperature changes have no effect on the original steel stress, since the tension members and the compression member are of almost identical length. The steel prestress is about 120,000 to 140,000 psi, based on commercially available high-carbon wire, in diameters of about .08 to .11 in., and a minimum yield point of about 180,000 to 210,000 psi.

The slender center member is subjected to a compressive stress of about 100,000 psi. Buckling is prevented due to the radial reactions exerted by the reinforcing wires over their spacer supports. The effective column length is thereby limited to the distance between adjacent spacers, so that the load capacity is independent of the total length of the prestress unit.

*U. S. Patent No. 2,303,394 and pending domestic and foreign applications.

Fig. 2(a) (lower left) and 2(b) (at right)—Reinforcing wires connected to end members

Fig. 1—Self contained, prestressed reinforcing units

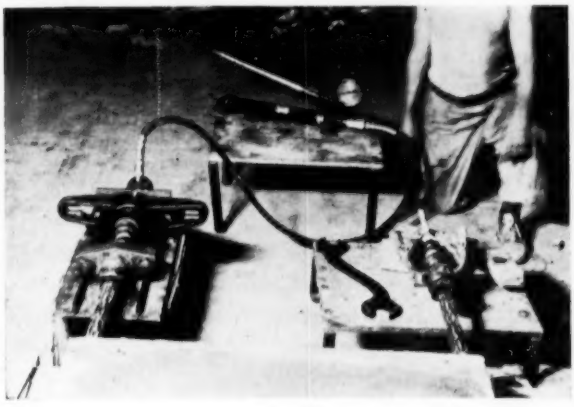


Fig. 3—After the concrete is cured and hardened, the compression rod is released by means of end attachments

A theoretical analysis indicates that the degree of elastic stability at the column supports is mainly a function of the helical pitch, or braiding angle of the wires and, to a lesser extent, of their stress.

Fig. 1 shows that the elastic force system also remains stable in case of bending. A relatively small radius of curvature introduces considerable secondary stresses. Bent-up units, with a relatively large radius of curvature, are advantageous in beam and girder construction to reduce the so-called "diagonal tension" near the supports.

The loaded units are placed in ordinary wooden forms, in the same manner as reinforcing bars, and thoroughly embedded in a rich mix of concrete mortar. The side forms are stripped early, so that possible surface defects can be repaired promptly.

Upon proper curing and hardening of the concrete, the compression rod is gradually released by means of attachments, such as shown in Fig. 3. The steel stretching force reaction is thereby transferred to the concrete in the form of internal compression.

The protruding end members and the center rod are then removed, thus assuring the transfer of the entire prestress load to the concrete. The projecting end wires are cut or bent inside the face of the concrete and protected with mortar. If necessary, the hole left by the compression rod can be grouted throughout its length. The end members and the compression rod are reused in the fabrication process.

The proposed construction method requires the standardized fabrication of reinforcing units in different sizes, with load capacities from about 2 to 20 tons per unit. The prestress units, as shown in Fig. 1 and 2(a), have a capacity of about 10 tons, with an outside diameter of $1\frac{5}{8}$ in.

Considerable improvements in size and capacity will become possible with high yield point wire having the properties reported by Hoyer, in connection with SAE 4140 compression members.

6—BOND

The bond of plain reinforcing bars, embedded in concrete without initial tension, is generally explained by adhesion or grip and frictional resistance. The initial slip appears to be almost identical for both plain and deformed bars.

A discussion of bond developed by smooth bars is given by Gilkey,[12] Chamberlin and Beal. The initial slip is probably influenced by a slight contraction of the bar, determined by Poisson's ratio. The imposed tensile stress develops a tendency for the bar to pull away from the surrounding concrete, whereby the initial resistance to sliding is reduced. The additional resistance then mainly depends on the surface conditions and initial bar deformations.

Prestressed reinforcement is embedded in concrete under an initial tensile stress. Upon release of the prestress force this stress is reduced, due to the immediate elastic shortening of the concrete and subsequent volume changes. Under this condition the wire develops a tendency to enlarge its diameter slightly, again in accordance with Poisson's ratio. As reported by Hoyer, such bond is reliable for straight wires of about ⅛-in. maximum diameter.

This action is similar in effect to that of a "forced fit" or "press fit," where considerable resistance to sliding is developed in spite of the smooth contact surfaces. The "forced bond" disappears in case the steel stress again approaches its original value, as indicated by some pull-out tests made in St. Louis.

Prestressed reinforcing units provide additional slip resistance due to the helical shape of the wires. In case of larger diameters, it may be expected that the release of a relatively high steel prestress will exert an excessive radial pressure on the concrete, which may cause splitting or spalling, especially when the reinforcement is near the surface.

7—VOLUME CHANGES

The volume changes of concrete, as here defined, are the time deformations due to the effect of moisture conditions (shrinkage and swelling) and the effect of sustained loads (plastic flow or creep). For the present purpose each component is treated as an independent subject, although the time deformation of prestressed members includes the simultaneous effect of moisture conditions, stress, changes in the elastic and thermal properties, etc.

(a) Shrinkage and swelling

A summary of various investigations is given by Davis.[13] Besides the initial water content of the mix, the relative shrinkage of concrete,

designated by e_s, is mainly due to changes in moisture conditions. By designating the relative humidity of the surrounding atmosphere, with h, expressed as a fraction, the following approximate relation is suggested for the ultimate shrinkage deformation of Ordinary Mixes and relatively thin members.

$$e_s = \frac{.125\ (.90 - h)}{100} \quad \ldots\ldots (8)$$

Assuming a relative humidity, $h = .50$, the shrinkage contraction becomes $e_s = .05$ per cent, corresponding to a temperature drop of about 83 F. For a saturated condition, (fog or water), $h = 1.00$, and $e_s = -.0125$ per cent (expansion). Reinforced concrete, subject to permantly moist storage conditions is therefore relatively free from shrinkage cracks.

(b) Plastic flow or creep

Plain concrete, subjected to sustained loads, will flow or creep at a diminishing rate with time. The results of various investigations have been summarized by Davis,[14] Shank,[15] and Morris.[16]

The plastic flow deformation, e_p, due to a sustained concrete stress, f_c, is expressed by the relation,

$$e_p = y\, f_c \quad \ldots\ldots (9)$$

In the determination of the flow per unit stress, designated by y, the humidity conditions seem to dominate other factors, such as mix, type of aggregate, etc. The following approximate relation is suggested for Ordinary Concrete, wet-cured up to the age at loading, α, and stored during a time, x, in air with the relative humidity, h, or water ($h = 1.00$),

$$y = \frac{.60}{1{,}000{,}000}\ (1.25 - h) \left(\frac{x}{\alpha}\right)^{.30} \quad \ldots\ldots (10)$$

The ratio, x/α, is defined as the relative flow period, or the relative age of a stressed member. The introduction of this age ratio greatly facilitates the comparison and interpretation of various plastic flow data on a common, relative time basis. The age at loading becomes the unit of time.

A test prism loaded at the age of 7 days is relatively old at the age of 3 months. An identical prism, loaded at the age of 3 months is relatively young at the age of one year. Under identical curing and storage conditions, the flow of both prisms will probably reach almost identical values at the ages of 5 months and 5 years respectively. The extended curing period of the second prism undoubtedly defers, but does not seem to decrease the long time flow.

In this connection, it is interesting to note from the observations reported by Pickett,[17] on the effect of drying on the bending of loaded beams, that identical curing and loading conditions result in almost

identical time deflections, when the latter are referred to the age ratio, x/α, instead of the actual age of the beam.

For design purposes, the flow can be considered as completed within a time period, $x = 10\alpha$, to 30α, for which limits, $(x/\alpha)^{.30} = 2.00$ and 2.77 respectively. The average value is 2.38, corresponding to a flow period, $x = 18\alpha$, which is substituted in equation (10) for the purpose of estimating the flow of Ordinary Concrete.

$$y = \frac{1.43}{1{,}000{,}000}(1.25 - h) \quad \ldots\ldots\ldots\

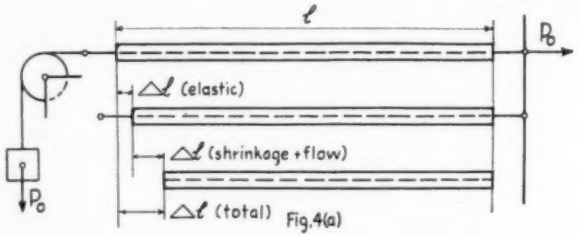


Fig.4(a)

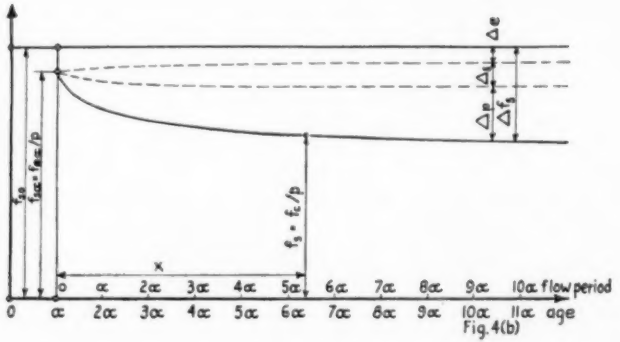


Fig.4(b)

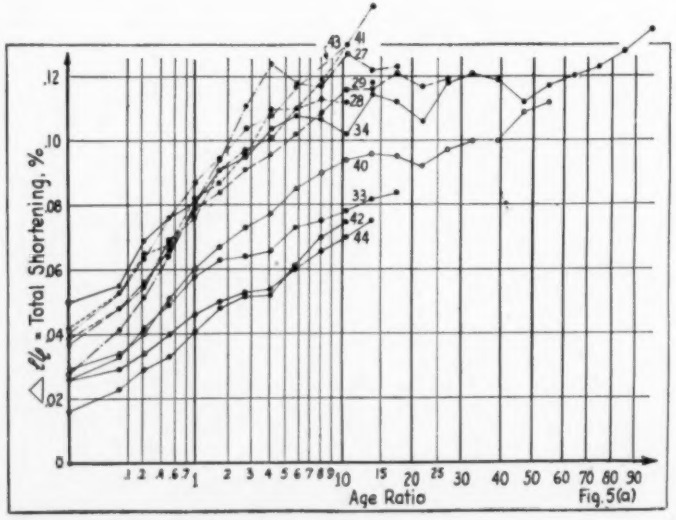


Fig.5(a)

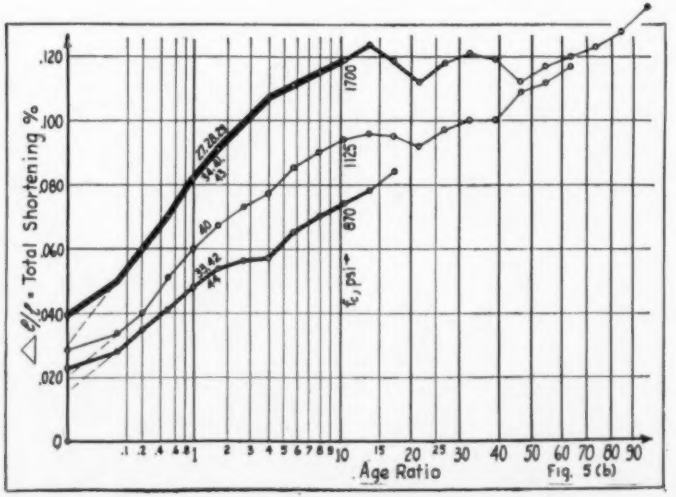


Fig. 5(b)

Fig. 4(a) top left; 4(b) top right
Fig. 5(a) bottom left; and 5 (b) bottom right

TABLE 1(a)—TEST PRISM DATA

Test Prism, No.	Dimensions, Inches	A_c In.²	A_s In.²	Reinforcing Wire, Dia. In.	Reinforcing Wire, Type	Concrete, Age (Days) at Release	Concrete, Age (Days) at Last Obs.	Concrete, Strength Days	Concrete, Strength psi	Atmosphere, Average Temp. F.	Atmosphere, Average Hum. %
27	3x3.2x140	9.1	.160	.113	(1)	7	131	6 28	4720 5730	56	35
28	3x3x140	8.5	.160	.113	(1)	11	123	7 11	3720 4490	54	35
29	3x3x126	8.5	.160	.113	(1)	4	127	2 4 28	2650 4090 5840	52	36
33	4.5x4.5x133	19.8	.160	.113	(1)	6	107	5	2900	50	37
34	3x3x140	8.5	.160	.113	(1)	4	438	4 14	3430 5560	52	34
40	3x3x126.5	8.7	.076	.078	(2)	7	399	7	4300	51	35
41	3x3x136	8.5	.160	.113	(1)	27	383	7 28	3380 5060	53	34
42	3.75x3.75x131	13.8	.076	.078	(2)	28	384	7 28	3380 5060	54	34
43	3x3x132.5	8.5	.160	.113	(1)	21	353	7 21	3690 5450	54	33
44	3.75x3.75x129.5	13.8	.076	.078	(2)	21	353	7 21	3690 5450	54	33
Average						14			(3) 4500	53	35

NOTES:
(1), Laclede Spring Wire, tensile strength = 208,500 psi, yield point 188,000 psi.
(2), Washburn Music Wire, tensile strength = 280,000 psi, yield point about 250,000 psi.
(3), At loading.

After a few days hourly readings were taken before removal of the center rod. The initial friction force for this operation was observed by means of a hydraulic jack gauge. A corresponding correction has been added to the initial deformation of the prism.

The ends of the prism were then cut off by means of a carborundum saw and the specimens transferred to a dry basement. A correction was also added for this time lag, based on the previous and subsequent hourly readings, after the specimens were placed on 10-in. I-beams in the storage room. One of the smooth ends was slightly pressed against the face of a steel angle fastened to the supporting beam, with Ames dials clamped to the same beam at the opposite end. Free movement of the observed end was facilitated by means of roller or hanger supports of the specimen.

TABEL 1(b)—TEST PRISMS, STRESS CONDITION

Test Prism No.	E_s, Million psi	At Release of Prestress Force, age α								At the age 11.6 α			
		$\Delta l/l$ Per Cent	Δf_s psi	f_{s0} psi	f_s psi	p Per Cent	f_c psi	E_c(1) Million psi	n	$\Delta l/l$ Per Cent	Δf_s psi	f_s psi	f_c psi
27	28	.0285	8,000	124,500	116,500	1.76	2060	7.2	3.9	.127	35,600	88,900	1570
28	28	.0436	12,200	124,500	112,300	1.87	2100	4.8	5.8	.112	31,400	93,000	1740
29	28	.0372	10,500	124,500	114,000	1.87	2130	5.7	4.9	.116	32,500	92,000	1720
33	28	.0267	7,500	124,500	117,000	.81	950	3.5	7.9	.078	21,800	102,700	830
34	28	.0501	14,000	124,500	110,500	1.87	2060	4.1	6.8	.102	28,600	95,900	1790
40	30	.0300	9,000	156,200	147,200	.87	1320	4.4	6.8	.094	26,300	128,200	1120
41	28	.0386	10,800	127,000	116,200	1.87	2170	5.6	5.0	.128	35,800	91,200	1710
42	30	.0166	5,000	183,000	178,000	.55	980	5.9	5.1	.075	21,000	160,500	890
43	28	.041	11,500	124,500	113,000	1.87	2110	5.2	5.4	.130	36,400	88,100	1650
44	30	.0257	7,700	183,000	175,300	.55	970	3.8	7.9	.070	19,600	162,000	890
Average								5.0	5.9				

NOTE: (1), Individual values uncertain due to difficulties in observing the instantaneous deformations and determination of friction force of compression rod.

It was assumed, at first, that temperature changes would be compensated due to identical thermal properties of the steel beam and test specimen. However, an examination of the observed temperatures indicated a differential movement corresponding to a value of about +.03 per cent per 100 F. temperature change. This may be due to a partial restraint of the supporting steel beam or a difference in the coefficients of expansion. A temperature correction, corresponding to the above value, was therefore applied to the observed movements, based on the starting temperature. This correction is relatively small. The relative humidity in the storage room was observed by readings from a Taylor Humidguide. All readings were made twice daily until the changes became small enough to permit observations at longer time intervals of one week max. No corrections were applied for changes in the relative humidity, although the plotted curves indicate an almost immediate effect due to this cause, especially after the main volume changes have been completed.

Fig. 5(a) shows all observations reduced to the same relative time basis on a cube root scale. The points shown represent individual values from the original curve, not averages over the given relative time interval, where the humidity changes would be much less pronounced.

The upper group, consisting of prisms No. 27, 28, 29, 34, 41 and 43, includes loading ages varying between 4 and 27 days. The average concrete stress of this group, at the age ratio 10.6, is 1700 psi. The lower group, consisting of specimens No. 33, 42 and 44, includes loading ages from 6 to 28 days. For the same age ratio the average stress of the lower group is 870 psi. Specimen No. 40, between these two groups, has a loading age of 11 days and a concrete stress of 1125 psi at the above age ratio.

It will be noted that the reduction to identical age ratios results in almost identical deformations for all specimens subjected to identical stress conditions. Fig. 5(b) shows the averages of the upper and lower group, also the individual points of No. 40. The upper group indicates a straight line relation up to a relative age of about 4. A similar behavior is suggested by the lower group, and even by the individual curve No. 40. The approach of a more or less stable condition begins at a relative age of about 10. However, the influence of humidity fluctuations and the average of fewer and fewer points only discloses a very general trend.

It seems that the deformations may increase very slowly for an almost indefinite time. The approach of an "ultimate deformation" at an age ratio of about 10 to 30 then would simply indicate a transition stage in the time law.

The initial trend of the curves is probably not correct, due to the evident difficulties in obtaining reliable observations on the immediate elastic deformation, without including plastic flow components. The release of the prestress force did not permit the instantaneous readings required for this purpose. For this reason the initial behavior is probably expressed by the dotted extension of the straight line range.

8—UNIFORMLY PRESTRESSED MEMBERS

(a) Steel stress loss

With reference to Fig. 4(b), the total steel stress reduction, Δf_s, is composed of an elastic stress component, Δ_e, a stress loss due to plastic flow, Δ_p, and a shrinkage stress loss, Δ_s.

Assuming an average value, $E_s = 28{,}000{,}000$ psi for high yield point steel wire, the steel stress loss due to shrinkage may be estimated from equation (8),

$$\Delta_s = 35{,}000\ (.90 - h) \quad \ldots\ldots (12)$$

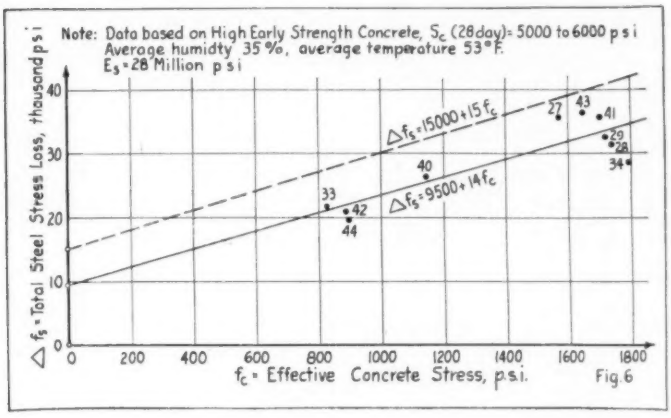
With relative humidities of 30, 50 and 70 per cent, $\Delta_s = 21{,}000$, 14,000 and 7,000 psi respectively.

From equations (4), (9) and (11), the elastic and plastic stress components are,

$$\Delta_e = nf_c \quad \ldots\ldots (13)$$

and,

Fig. 6


Note: Data based on High Early Strength Concrete, S_c (28 day) = 5000 to 6000 p s i
Average humidty 35 %, average temperature 53° F.
E_s = 28 Million p s i
Δf_s = Total Steel Stress Loss, thousand p s i
$\Delta f_s = 15000 + 15 f_c$
$\Delta f_s = 9500 + 14 f_c$
f_c = Effective Concrete Stress, p.s.i.
Fig. 6

$$\Delta_p = mf_c \quad \ldots\ldots (14)$$

where, for ordinary concrete,

$$m = 40\ (1.25 - h) \quad \ldots\ldots (15)$$

With relative humidities of 30, 50 and 70 per cent, m = 38, 30 and 22 respectively.

The concrete prestress has a maximum value at the age α, and approaches a constant value at the end of the flow period. For design purposes, f_c, in equations (13) and (14), is defined as the effective concrete stress at the end of the flow period. The total steel stress reduction then becomes,

$$\Delta f_s = \Delta_s + (n + m)\, f_c \quad \ldots\ldots (16)$$

Equations (12) to (16) inclusive are approximate expressions for estimating purposes.

Fig. 6 shows the total steel stress loss of the test prisms, Fig. 5(a), in function of the effective concrete stress at the relative age, x/α = 10.6. For this purpose the steel stresses are based on a uniform modulus of elasticity, E_s = 28,000,000 psi.

The average of all points has been derived by means of the method of least squares, assuming a straight line relation. The intersection with the ordinate, $f_c = 0$, gives the stress loss due to shrinkage (9500 psi), although such an extension is probably not justified. With due allowance for this uncertainty and somewhat increased plastic deformations at a later age, equation (6) is stated in the following tentative form for high early strength concrete,

$$\Delta f_s = 15{,}000 + 15\, f_c \quad \ldots\ldots (17)$$

The average value, E_c, as shown in Table (1), is in reasonable agreement with E_c derived from equation 3(b). It would seem that values, n = 5, and m = 10 are justified. However, for design purposes, n, and m, in equation (17), are tentatively assumed as follows:

$$n + m = 7.5 + 7.5 = 15 \quad (18)$$

For ordinary concrete, with average humidity conditions, ($h = .50$) from equation (15), and a minimum value, n, as above,

$$(n + m) = 7.5 + 30 = 37.5 \quad (19)$$

This comparison seems to indicate that the use of high early strength concrete results in a smaller steel stress loss. A comprehensive investigation of this subject, based on parallel tests, is evidently needed.

(b) Effective stresses

From equation (16), the effective steel stress, at the end of the flow period, is,

$$f_s = f_{so} - \Delta f_s = (f_{so} - \Delta_s) - (n + m) f_c \quad (20)$$

From equilibrium conditions,

$$f_c = \frac{A_s}{A_c} f_s = p f_s \quad (21)$$

By substituting above expression in equation (20),

$$f_s = \frac{(f_{so} - \Delta_s)}{1 + (n + m)p} \quad (22)$$

From equation (21),

$$f_c = \frac{p\,(f_{so} - \Delta_s)}{1 + (n + m)p} \quad (23)$$

The maximum effective stresses, immediately after release of the prestress force, with $m = 0$, become,

$$f_s\alpha = \frac{f_{so}}{1 + np} \quad (24)$$

and,

$$f_c\alpha = \frac{p f_{so}}{1 + np} \quad (25)$$

As an example, consider the test prism, No. 34. With $p = .0187$, $f_{so} = 125{,}000$ psi, from equations (17), (22) and (23),

$$f_s = \frac{125{,}000 - 15{,}000}{1 + 15 \times .0188} = 85{,}700 \text{ psi}$$

$$f_c = .0188 \times 85{,}700 = 1610 \text{ psi}$$

Table (2) shows the influence of a possible variation of ± 33.3 per cent for the steel stress loss, equation (17), combined with various assumptions for the original steel stress. It will be noted that a low original steel stress produces a low, and rather uncertain concrete prestress, combined with an inefficient use of steel. Besides, the elastic elongations of the steel are relatively small and require a fine adjustment in the stretching device.

TABLE 2—INFLUENCE OF STEEL PRESTRESS AND VOLUME CHANGES ON THE EFFECTIVE CONCRETE STRESS

(Uniform Prestress)

A_c In.²	P_o Lbs.	A_s In.²	p Per Cent	f_{so} psi	Δ_s psi	$n+m$	Δf_s psi	f_s psi	f_c psi	f_s, f_c Per Cent
				200,000	10,000	10	30,000	170,000	2000	108
8.5	20,000	.10	1.18	200,000	15,000	15	42,700	157,300	1850	100
				200,000	20,000	20	54,400	145,600	1720	93
				125,000	10,000	10	28,200	96,800	1820	113
8.5	20,000	.16	1.88	125,000	15,000	15	39,200	85,800	1610	100
				125,000	20,000	20	48,600	76,400	1430	89
				50,000	10,000	10	22,800	27,200	1280	132
8.5	20,000	.40	4.71	50,000	15,000	15	29,500	20,500	970	100
				50,000	20,000	20	34,600	15,400	730	75

TABLE 3—DETERMINATION OF THE EFFECTIVE STRESSES, METHOD OF SUCCESSIVE APPROXIMATIONS

(Uniform Prestress)

Example:

$A_s = .16$ in.², $A_c = 8.5$ in.², $p = .0188$, $P_o = 20{,}000$ lb., $f_{so} = 125{,}000$ psi, $\Delta_s = 15{,}000$ psi, $m = n = 7.5$, $\Delta f_s = 15\ (f_c + 1000)$

$P = .16 \times f_s$	$f_c = \frac{P}{8.5}$	$\Delta f_s =$	$f_s = 125{,}000 - \Delta f_s$
		0	125,000
20,000	2,350	50,300	74,700
12,000	1,410	36,100	88,900
14,200	1,670	40,000	85,000
13,600	1,600	39,000	86,000
13,800	1,620	39,300	85,700
13,700	1,610		

In contrast, a high original steel prestress produces a high, and reliable concrete prestress, obtained with a small amount of steel. In this case, the steel elongations are comparatively large, therefore easier to adjust and maintain.

The effective stress condition can also be determined by means of successive approximations, as shown in Table (3). For this purpose, the entire original prestress force, P_o, is at first assumed to be acting, whereby an approximate value for the concrete prestress is obtained. The corresponding steel stress reduction, from equation (16), then determines an

approximate value of the effective steel stress. The latter is used to obtain a second approximation for the effective prestress force and the procedure is repeated until the corrections become negligible.

(c) Axial loads

A uniform prestress condition is advantageous for members subjected to direct, axial loads, such as columns, hangers, ties and struts in truss construction, thrust ties in arches, etc. The application of the axial force, F, causes a concrete stress change, f_{cF}, and a steel stress change, f_{sF}, in the effective prestress system. From equilibrium conditions,

$$F = f_{cF} A_c + f_{sF} A_s \quad \ldots\ldots (26)$$

Since, $f_{sF} = n f_{cF}$,

$$f_{cF} = \frac{F}{A_c} \times \frac{1}{(1 + np)} \quad \ldots\ldots (27)$$

and,

$$f_{sF} = \frac{F}{A_c} \times \frac{n}{(1 + np)} \quad \ldots\ldots (28)$$

The stress changes due to the axial load then are superimposed on the effective prestress system.

As an example, assume a direct tensile force, $F = -15{,}000$ lb., applied to the prestressed prism, Table (3). From equations (27) and (28), with $n = 6$,

$$f_{cF} = \frac{-15{,}000}{8.5\ (1 + 6 \times .0188)} = -1590 \text{ psi} \quad \text{(tension)}$$

$$f_{sF} = 6 \times 1590 = +9500 \text{ psi}$$

With reference to Table (3), the resulting stresses become,

$$f_c = +1610 - 1590 = 20 \text{ psi}$$

$$f_s = 85{,}700 + 9500 = 95{,}200 \text{ psi}$$

The prestressed prism can therefore support an axial tensile force, equivalent to 75 per cent of the original prestress force, without causing tension in the concrete. Assuming a minimum compressive strength, $S_c = 3600$ psi, cracks may be expected to develop, from equation (1), when the pull exceeds about 18,000 lbs. The corresponding steel stress, about 97,300 psi, still is smaller than the original steel stress.

The stress changes due to axial loads can also be determined by the method of successive approximations, as shown in Table (4). As a first approximation, the entire axial load is assumed to be acting on the concrete, whereby an approximate value for the steel stress change is obtained. The corresponding change of the prestress force, Δ_P, gives a correction for the concrete stress change, etc.

If the external load consists of an axial compression force, $F = +15{,}000$ lbs., the combined stresses become,

TABLE 4—STRESS CHANGE DUE TO AXIAL LOAD, METHOD OF SUCCESSIVE APPROXIMATIONS

(Uniform Prestress)

Example:

$F = -15{,}000$ lb., $n = 6$, $A_s = .16$ in.^{2}, $A_c = 8.5$ in.^{2}

$P = f_{sF} \times A_s$	$f_c\Delta_P = \Delta P/A_c$	F/A_c	f_{cF}	$f_{sF} = n \times f_{cF}$
—	—	−1765	−1765	−10,600
+1700	+200	−1765	−1565	− 9,400
+1500	+175	−1765	−1590	− 9,500

$f_c = +1610 + 1590 = +3200$ psi (compression)

$f_s = 85{,}700 - 9500 = 76{,}200$ psi

The resulting steel stress still is a tensile stress. The area of the prism must be increased, in order to safely carry the above load as a column or a strut.

Observations on prestressed beams indicate a straight line relation between load and deflections over, and beyond, the entire range of normal loads. With an apparently constant modulus of elasticity for prestressed concrete, it can be expected that relatively long, prestressed columns, having a minimum age of about 10α, will fail more in accordance with Euler's equation. In case of hinged ends, with E_c as determined from bending tests, the ultimate load then should approach the value,

$$P_{cr} = \frac{\pi^2 E_c I}{L^2} \quad \ldots\ldots (29)$$

The actual behavior of long and short prestressed columns must first be determined by tests.

(d) Bending moments

As a rule, members subject to direct axial loads are also subjected to secondary bending stresses, due to rigid end connections, eccentric load applications, etc.

In case of symmetrical sections, and a symmetrical distribution of the steel area, as usually found in columns, the stress changes due to bending moments can be derived from the transformed-section method.

As an example, consider the rectangular section shown in Fig. 7, which is uniformly prestressed. The stress changes, due to an external moment, M, acting in the plane, $2 - 2$, are derived from the moment of inertia, $I_{1\text{-}1}$, of the transformed section,

$$I_{1\text{-}1} = \frac{b\,t^3}{12} + (n - 1)\,r^2\,A_s \quad \ldots\ldots (30)$$

or,

Fig. 7

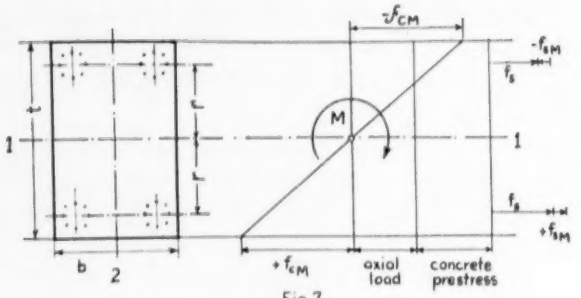


Fig.7

$$I_{1\text{-}1} = \frac{b\,t^3}{12}\left(1 + \frac{12\,(n-1)\,pr^2}{t^2}\right) \quad \ldots\ldots (31)$$

The section modulus for the extreme fiber is,

$$S = \frac{b\,t^2}{6}\left(1 + \frac{12\,(n-1)\,pr^2}{t^2}\right) \quad \ldots\ldots (32)$$

The concrete stress change, $f_{c\mathrm{M}}$, at the extreme fiber becomes,

$$f_{c\mathrm{M}} = \pm\frac{M}{S} = \pm\frac{6M}{b\,t^2} \times \frac{1}{1 + 12\,(n-1)\,pr^2/t^2} \quad \ldots\ldots (33)$$

The steel stress change, $f_{s\mathrm{M}}$, is,

$$f_{s\mathrm{M}} = \pm\frac{2nr}{t} f_{c\mathrm{M}} \quad \ldots\ldots (34)$$

9—BEAMS AND GIRDERS

(a) Prestress load

Fig. 8 shows a beam section, with an effective prestress force, P, acting at the distance, e, from the *c.g.c.* The average, effective concrete stress is,

$$f_{coP} = \frac{P}{A_c} \quad \ldots\ldots (35)$$

The eccentric application causes an internal moment,

$$M_P = e\,P \quad \ldots\ldots (36)$$

The extreme fiber stresses are,

$$f_{c1P} = \frac{P}{A_c} + \frac{eP}{S_{c1}} \quad \ldots\ldots (37)$$

$$f_{c2P} = \frac{P}{A_c} - \frac{eP}{S_{c2}} \quad \ldots\ldots (38)$$

By designating,

$$s_1 = \frac{S_{c1}}{A_c} \quad \ldots\ldots (39)$$

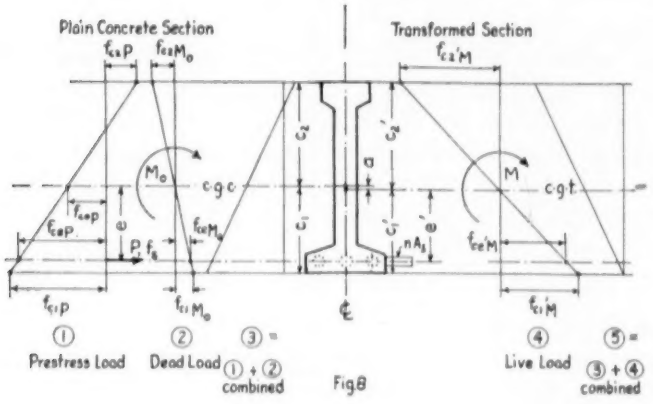


Fig. 8

and,

$$s_2 = \frac{S_{c2}}{A_c} \quad \ldots\ldots (40)$$

the fiber stresses then are determined by means of the stress ratios,

$$k_1 = \frac{f_{c1P}}{f_{coP}} = \left(1 + \frac{e}{s_1}\right) \quad \ldots\ldots (41)$$

and,

$$k_2 = \frac{f_{c2P}}{f_{coP}} = \left(1 - \frac{e}{s_2}\right) \quad \ldots\ldots (42)$$

The concrete stress at the *c.g.s.* is,

$$f_{ceP} = f_{c1P} - (f_{c1P} - f_{c2P}) \frac{(c_1 - e)}{(c_1 + c_2)} \quad \ldots\ldots (43)$$

After a few transformations,

$$f_{ceP} = f_{coP}\left(1 + \frac{e^2 A_c}{I_c}\right) \quad \ldots\ldots (44)$$

or, also, by introducing the stress ratio,

$$k_e = \frac{f_{ceP}}{f_{coP}} = \frac{I_e}{I_c} \quad \ldots\ldots (45)$$

The effective steel stress is,

$$f_{sP} = \frac{P}{A_s} \quad \ldots\ldots (46)$$

The originally required steel stress can now be determined by substituting, f_{ceP}, in equation (16).

The designer is confronted with the opposite problem, i.e. the determination of the effective stresses due to the release of the original prestress force, P_o, also the simultaneous influence of dead loads.

(b) Dead load

If the dead load moment, M_o, is smaller than the counteracting prestress moment, equation (36), the release of the prestress force will cause the beam to lift off the supporting forms. In this case the dead load acts simultaneously with the prestress load and the steel stress loss is determined by the combined concrete stress at the *c.g.s.* The concrete stress, f_{ceM_o}, due to the dead load moment, is

$$f_{ceM_o} = \frac{M_o}{S_e} \quad \ldots\ldots (47)$$

The effective stresses are preferably determined by the method of successive approximations. For this purpose the known value, P_o, instead of P, is at first substituted in equation (35), which results in an approximate stress, f_{ceP}, as derived from equation (45). This stress is combined with the given dead load stress, equation (47). The resulting stress, substituted in equation (16), gives an approximation for the total steel stress reduction, therefore a correction for the effective prestress force. The further procedure is similar to that of Table (2).

The effective fiber stresses are now obtained from equations (41) and (42), by means of the final value of P, as substituted in equation (35). In case of top and bottom reinforcement the corrections should be made simultaneously, although the two prestress forces can be treated independently as a first approximation.

(c) Live load

The stress changes due to live loads are based on the transformed-section, assuming that volume changes can be considered as completed.

The transformed area, A, is based on the entire concrete section,

$$A = A_c + n\,A_s \quad \ldots\ldots (48)$$

With reference to Fig. (8), the location of the neutral axis is given by the relation,

$$a = \frac{e\,n\,A_s}{A} \quad \ldots\ldots (49)$$

$$I = I_c + a^2 A_c + (e - a)^2\,n\,A_s \quad \ldots\ldots (50)$$

or, also,

$$I = I_c + a\,e\,A_c \quad \ldots\ldots (51)$$

with $c'_1 = c_1 - a$, the section modulus for the bottom fiber becomes,

$$S'_{c1} = \frac{I}{c'_1} \quad \ldots\ldots (52)$$

For the top fiber, with $c'_2 = c_2 + a$,

$$S'_{c2} = \frac{I}{c'_2} \quad \ldots\ldots (53)$$

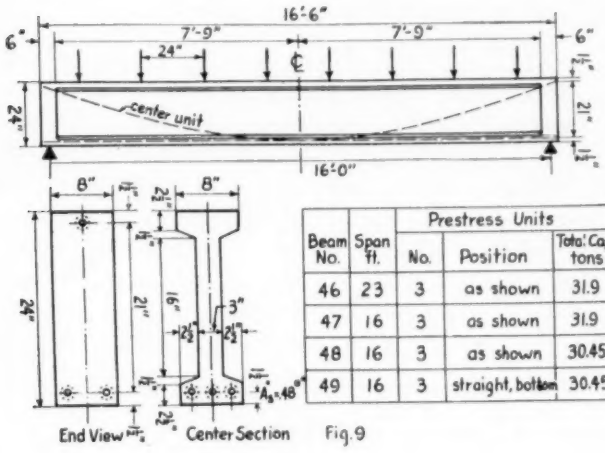


Fig. 9

Beam No.	Span ft.	Prestress Units		
		No.	Position	Total Cap. tons
46	23	3	as shown	31.9
47	16	3	as shown	31.9
48	16	3	as shown	30.45
49	16	3	straight, bottom	30.45

Fig. 9

For the *c.g.s.*, with $e' = e - a$,

$$S'_e = \frac{I}{e'} \quad \ldots\ldots (54)$$

The concrete stress changes produced by the live load moment, M, then are,

$$f'_{c\,1M} = \frac{M}{S'_{c\,1}} \quad \ldots\ldots (55)$$

$$f'_{c\,2M} = \frac{M}{S'_{c\,2}} \quad \ldots\ldots (56)$$

$$f'_{c\,eM} = \frac{M}{S'_e} \quad \ldots\ldots (57)$$

and the steel stress change, from equation (4),

$$f_{sM} = n\, f'_{c\,eM} \quad \ldots\ldots (58)$$

(d) Example, working stresses

Fig. 9 shows the dimensions of a simple test beam, designed for a uniform live load of 2000 lb./lin. ft. The reinforcement consists of three prestress units, of which the center unit is bent-up as shown. Table (5) gives the properties of the center section.

Table (6) shows the determination of the effective steel stress, by means of the method of successive approximations, based on the assumption that no other dead loads than the weight of the beam will act during the period of volume changes. Table (7), column (3), shows the resulting fiber stresses, which are all based on the plain concrete section.

The stress changes due to live loads are derived from the transformed-section. The resulting bottom fiber stress under the normal live load,

TABLE 5—PROPERTIES OF BEAM SECTION

(Fig. 9, center of span)

Section	Reference	Distance In.	Area In.^{2}	Moment of Inertia In.^{4}	Section Modulus In.^{3}	Stress Ratio
Plain Concrete	top	$c_2 = 11.84$			$S_{c2} = 573$	$k_2 = -.91$
	c.g.c.	0	$A_c = 102.9$	$I_c = 6,790$		
	c.g.s.	$e = 10.66$	$A_s = .48$	$I_s = 18,480$	$S_s = 637$	$k_s = 2.72$
	bottom	$c_1 = 12.16$			$S_{c1} = 558$	$k_1 = 2.97$
Transformed	top	$c'_2 = 12.2$			$S_{c'2} = 590$	
	n.a.	0	$A = 106.5$	$I = 7,180$		
	c.g.s.	$e' = 10.3$			$S'_s = 700$	
	bottom	$c'_1 = 11.8$			$S_{c'1} = 610$	

TABLE 6—EFFECTIVE STEEL STRESS

(Fig. 9, center of span (16 ft.-0 in.))

$\Delta_s = 15,000$ psi, $m = n = 7.5$, $P_o = 63,750$ lb., $f_{so} = 133,000$ psi, $M_o = 41,100$ in. lbs.

P	f_{soP}	f_{csP}	f_{csM_o}	$f_{cs(M_o + P)}$	Δf_s	f_s
						133,000
63,750	+620	+1685	−65	+1620	39,300	93,700
45,000	+437	+1190	−65	+1125	31,800	101,200
48,500	+470	+1280	−65	+1215	33,200	99,800
47,900	+465	+1265	−65	+1200	33,000	100,000

TABLE 7—FIBER STRESSES (PSI.)

(Fig. 9, center of span. (16 ft.-0 in.)

$M = 768,000$ in. lbs.

Location	(1) Prestress Force	(2) Dead Load	(3) = (1) + (2)	(4) Live Load	(5) = (3) + (4)	(6) ½ Live Load	(7) = (5) + (6)
Top	− 425	+75	− 350	+1300	+ 950	+ 650	+ 1600
C.g.c.	+ 465	0	+ 465				
C.g.s.	+1265	−65	+1200	−1100	+ 100	− 550	− 450
C.g.s. (steel stress)			100,000	8200	108,200	4100	112,300
Bottom	+1385	−75	+ 1310	−1260	+ 50	− 630	− 580

column (5), still is compression. The rated capacity of the beam should be limited to a zero stress at this point.

In order to obtain some indication for the factor of safety against cracking, it will be advisable to apply an excessive live load, equivalent to, say, 50 per cent of the normal live load, as shown in column (7). The resulting bottom fiber stress should not exceed the flexural strength of the concrete at the age of, say, 28 days. The minimum factor of safety

against cracking will then be 1.5, based on the previous limit under a normal live load.

Due to the small dead load—live load ratio (less than 6 per cent) the release of the prestress force produces an immediate tensile stress of about 380 psi at the top fiber, which, in time, will be reduced to the value indicated by column (3). This dead load stress should not exceed, say, 50 per cent of the flexural strength of the concrete at the age of release. It will be noted from the tests described below that non-uniform shrinkage during the early drying out period may cause a considerable increase in this stress. In order to avoid top cracks, it will therefore be advisable to apply a temporary load during this transition period. Precast members are generally provided with some top reinforcement, so as to permit safe handling and transportation.

The compressive stress at the age of release should be limited to about 40 to 50 per cent of the cylinder strength at that age. Under live load conditions, the maximum compressive stress may be based on the 28 day strength. The steel stress increase is of minor importance under all normal load conditions, since the original value is not reached again.

The ultimate load capacity may be derived from the assumption of a cracked tension zone, similar to the procedure used in the design of ordinary reinforced concrete beams. Tests carried out so far indicate that the above stress limitations will result in a minimum factor of safety of two under the normal live load, assuming that the shearing stresses will not determine the ultimate load capacity.

10—SHEARING STRESSES AND PRINCIPAL STRESSES

With reference to Fig. 10, the shearing stress at any axis, T-T, above or below the neutral axis, is given by the general relation,

$$v = \frac{V\,Q}{t\,I} \qquad (59)$$

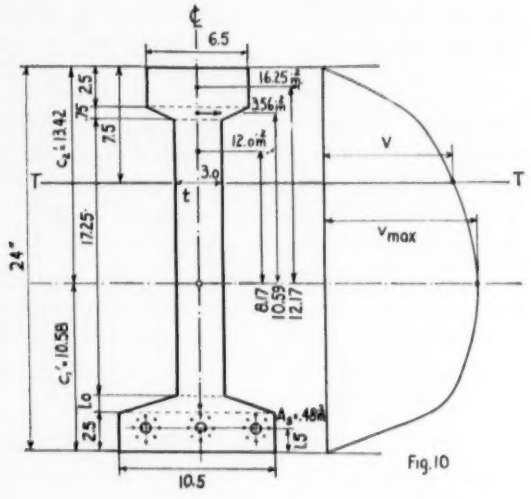
In the above expression, V represents the total shear, Q designates the statical moment of the area above or below that axis, about the neutral axis, and, I, designates the moment of inertia of the entire transformed-section. The maximum shearing stress, designated by v_{max}, occurs at the neutral axis.

As an example, from Fig. 10, with T-T located 7.5 in. below the top fiber,

$$Q = 16.25 \times 12.17 + 3.56 \times 10.59 + 12 \times 8.17 = 333 \text{ in.}^3$$

The moment of inertia of the transformed-section is, $I = 7100$ in.^{4}, therefore,

Fig. 10



$$v = \frac{V \times 333}{3 \times 7100} = \frac{V}{64}$$

At the neutral axis,

$$v_{max} = \frac{V}{54}$$

For approximate purposes, it may be assumed that the entire shearing force is carried by the web section of the beam, with a parabola shape stress distribution. In this case, for the above example,

$$v_{max} = \frac{V}{3 \times 24} \times \frac{3}{2} = \frac{V}{48}$$

Concrete is a brittle material, which fails either in compression or tension, similar to cast iron. The shearing stress is therefore not a reliable indication for the critical stress condition near the supports, except in special cases, such as ordinary concrete beams.

The prestress action requires a determination of the principal tensile stress, which is the cause of so-called "diagonal tension" failures.

With reference to Fig. 11, which shows the positive directions of stress, the principal stresses, S_1 and S_2, at any point, x, y, are given by the well known expressions,

$$S_1 = \frac{S_x + S_y}{2} + \frac{1}{2}\sqrt{(S_x - S_y)^2 + 4v^2} \quad \ldots\ldots (60)$$

$$S_2 = \frac{S_x + S_y}{2} - \frac{1}{2}\sqrt{(S_x - S_y)^2 + 4v^2} \quad \ldots\ldots (61)$$

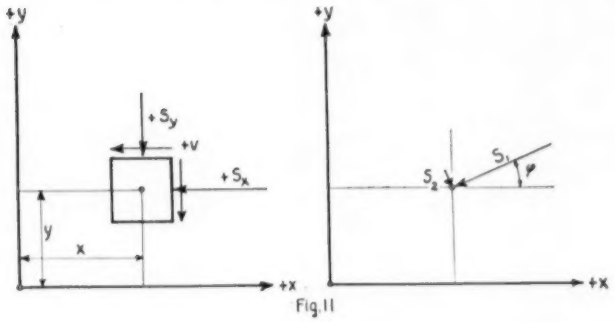


Fig. 11

Fig.11

S_x and S_y designate the normal stress components acting on a small element, compression being assumed positive. The shearing stress component, v, is of identical magnitude in both directions.

Equations (60) and (61) indicate that S_1 and S_2 are independent of the sign of v, which only influences the direction of the principal stress planes, defined by the angle φ,

$$\tan 2\varphi = \frac{2v}{S_x - S_y} \quad \ldots\ldots (62)$$

The following stress ratios are introduced,

$$p_x = \frac{S_x}{v} \quad \ldots\ldots (63)$$

$$p_y = \frac{S_y}{v} \quad \ldots\ldots (64)$$

$$p_1 = \frac{S_1}{v} \quad \ldots\ldots (65)$$

$$p_2 = \frac{S_2}{v} \quad \ldots\ldots (66)$$

Equations (60) and (61) can now be written in the form,

$$(p_x - p_1) \times (p_y - p_1) = 1 \quad \ldots\ldots (67)$$

and,

$$(-p_x + p_2) \times (-p_y + p_2) = 1 \quad \ldots\ldots (68)$$

By adding equations (60) and (61), as a check,

$$p_x + p_y = p_1 + p_2 \quad \ldots\ldots (69)$$

From equation (62),

$$\tan 2\varphi = \frac{2}{p_x - p_y} \quad \ldots\ldots (70)$$

Equations (67) and (68) indicate that constant values of p_1 and p_2, are located on equilateral hyperbolas, when p_x and p_y are the co-ordinates, as shown in Fig. 12. Assuming a positive sign for the shearing stress, the

Fig. 12

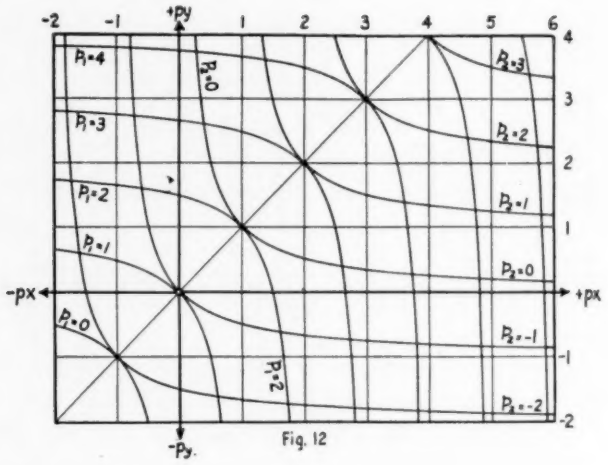


Fig. 12

area above and to the right of the curve, $p_2 = 0$, then indicates the compressive zone of principal stresses, expressed by the condition,

$$p_x \times p_y \geqq +1 \qquad (71)$$

It will be noted that the complete elimination of tensile stresses requires a compressive stress component not only in the longitudinal direction but also in the normal direction of the beam.

As an example, assume a simply supported prestressed girder, with an effective longitudinal stress component, $S_x = +600$ psi (compression), and a shearing stress, $v_{max} = +200$ psi, acting at the neutral axis over the supports. From equations (63) and (64),

$$p_x = \frac{+600}{+200} = +3.0$$

$$p_y = 0$$

From Fig. 12,

$$p_1 = +3.30$$

$$p_2 = -.30$$

As a check,

$$p_1 + p_2 = +3.0 = p_x + p_y$$

From equations (65) and (66), the principal stresses at this point then are,

$$S_1 = +3.30 \times 200 = +660 \text{ psi} \quad \text{(compression)}$$

$$S_2 = -\ .30 \times 200 = -\ 60 \text{ psi} \quad \text{(tension)}$$

Assume now that prestressed stirrups are added, which give an effective vertical stress component, $S_y = +100$ psi (compression), acting at the same point. In this case,

$$p_x = \frac{+600}{+200} = +\ 3.0$$

$$p_y = \frac{+100}{+200} = +.50$$

From Fig. 12,

$p_1 = +3.35$

$p_2 = +.15$

As a check,

$p_1 + p_2 = +3.50$

The principal stresses now become,

$S_1 = +3.35 \times 200 = +670$ psi (compression)

$S_2 = +.15 \times 200 = +30$ psi (compression)

The provision of a comparatively small vertical prestress has therefore eliminated the previous tensile stress at this point. Live loads and dead loads acting above the neutral axis also produce a vertical compressive stress component. Live loads applied below the neutral axis produce a tensile stress component in the vertical direction.

In ordinary reinforced concrete design, without prestress action in any direction,

$p_x = p_y = 0$

From Fig. 12

$p_1 = +1$

$p_2 = -1$

With a shearing stress, $v = +200$ psi, as before, the principal stresses are

$S_1 = +200$ lb. psi (compression)

$S_2 = -200$ lb. psi (tension)

In this special case the principal tensile stress is identical with the shearing stress.

It should be pointed out that the maximum principal tensile stress in prestressed beams is not necessarily found at the neutral axis. For this reason it is desirable to obtain longitudinal compressive stresses over the entire section near the supports.

11—BEAM TESTS AND DEFLECTIONS

The design principles, as outlined above, have been used as a guide in the design of test beams, reinforced with prestressed units, in connection with development work carried out in St. Louis, Mo. since 1939.

The preliminary observations were made on uniformly stressed, high early strength concrete prisms, of the type shown in Table 1, either loaded at the center or at the 1/3 points. Subsequent rectangular prisms, having a 3 x 6 in. section, were reinforced on the bottom side. Some of these specimens were provided with bent-up prestress units, giving a uniform compression over the entire end section.

The reinforcing steel consisted of cold drawn spring wire and music wire, with a diameter of .08 to .11 in., and stressed about 100,000 to 150,000 psi, corresponding about to a 2/3 yield point stress. The concrete prestress varied approximately between 1000 to 2000 psi.

The result of these tests may be summarized as follows: No bond or shear failure was observed, in spite of the absence of any special anchorage. Cracks became visible at a computed concrete tensile stress of about 800 to 1000 psi. The ultimate load was reached due to excessive cracks at the points of maximum moment, in some cases combined with crushing of the top fiber. No rupture of reinforcing wires has been observed, since the load was not increased beyond the stage of excessive cracks.

The relatively small deflections indicated a straight line relation between load and deflection beyond the range of compressive stresses. Sustained loads produced a time effect. The recovery of deflections was rapid and almost complete, even in case of loads which had produced visible cracks.

More recent test beams, of the type shown in Fig. 9, were intended to determine the influence of high shearing stresses. Fig. 13 shows the test arrangement and the result of these tests is summarized in Table 8.

The design stress analysis for the center section of beam No. 47, (16 ft. span), is given in Tables 5, 6 and 7. It will be noted that the suggested stress limitations result in a minimum factor of safety of 2.2 for this beam, since the load was not increased to produce complete failure.

With reference to equation (59), the rated load of this beam, together with the dead load, gives, $v_{max} = +320$ psi, over the supports. The effective longitudinal prestress component at the neutral axis is $S_x = +500$ psi. The uniformly distributed live and dead loads above the neutral axis, produce a vertical stress component, $S_y = +60$ psi. The corresponding principal tensile stress then becomes $S_2 = -110$ psi.

Under the maximum test load the stress components over the supports are, $v_{max} = +680$ psi, $S_x = +500$ psi and $S_y = +125$ psi. The corresponding principal tensile stress then is $S_2 = -390$ psi. Equation (1) gives an approximate tensile strength, $S_t = 360$ psi for the compressive strength of 5220 psi at the test age. The magnitude of the principal tensile stress indicates that a failure due to "diagonal tension" could be expected.

The fact that this beam did not fail in shear may be partly explained by the stress distributing effect of the vertical and rib, also by a small vertical prestress component induced by the bent-up prestress unit. At the same time it should be noted that the test load was distributed over 8 points, with the first point spaced 12 in. from the support.

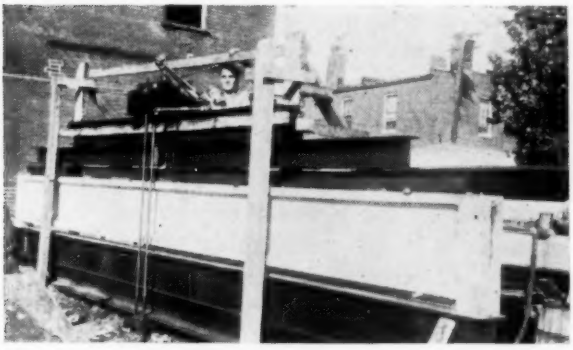


Fig. 13—Beam test reported in Table 8.

TABLE 8—BEAM TESTS

Test Beam No.	Span Ft.	Original Steel Stress f_{so} psi	Age of Beam		Compressive Strength		Live Load, and (v_{max})		
			At Release, Days	At Test, Days	Age, Days	S_c psi	Rated, Uniform Lbs.	Test, Max. Lbs.	Ratio
46	23	133,000	5	20	5 13	2790 3170	21,900 (220)	52,000 (500)	2.4
47	16	133,000	5	16	5 16	3440 5220	33,200 (320)	72,300 (680)	2.2
48	16	127,000	5	13	5 16	3000 3930	31,200 (300)	65,000 (610)	2.1
49	16	127,000	5	20	5 22	3000 4770	31,200 (300)	65,000 (610)	2.1

Remarks:

Dimensions and reinforcement as shown in Fig. 9, except for beam No. 49. Rated load capacity based on zero stress at bottom fiber (center). Test load distributed over 8 points, equally spaced.

No. 46. The actual center depth of this beam was 23.7 in. Max. test load produced 6 visible cracks, tension zone (center). No visible shear cracks.

No. 47. Max. test load produced 8 visible cracks, tension zone (center). No visible shear cracks.

No. 48. After release of the prestress force 2 cracks, about 10 in. deep, were observed near the top center. Test load applied December 3, 1942, air temperature 7 F., coal stove accidentally placed near one end of this beam. At the maximum load, 2 cracks were visible in the bottom tension zone (center), followed by a bond or shear failure of the heated end. No visible shear cracks at the opposite end.

No. 49. This beam was reinforced with three straight prestress units at the bottom. After release of the prestress force, one top crack was observed, about 20 in. from the end. The maximum test load produced 16 visible cracks in the tension zone (center). No visible shear cracks.

Tests No. 48 and 49 are not directly comparable on account of the initial top cracks. It appears that the end failure of beam No. 48 was influenced by the accidental heat exposure. This conclusion is supported by the behavior of the other end and the absence of visible shear cracks for beam No. 49. It is interesting to note that the latter did not fail in shear in spite of the initial top crack near one end.

These limited observations, and the experience of others, indicate that prestressed beams and girders will resist considerably higher shearing stresses than ordinary reinforced concrete beams, without resort to heavy shear reinforcement in many cases. Prestressed stirrups in deep beams and girders will be advisable, in order to assure monolithic action. The consideration of other load conditions, such as lateral moments, temperature stresses, etc., will also influence the amount of vertical reinforcement.

Fig. 14 shows the observed center deflections of beams No. 46 and 47. Fig. 15 shows the deflection diagram for an entirely different beam section and a comparatively long span. The reasonably straight line relation between load and deflection suggests a constant modulus of elasticity. The more or less pronounced "yield point" indicates that the flexural strength of the concrete has been reached, although cracks do not become immediately visible.

The instantaneous modulus of elasticity, for the straight line range, can be determined from the theory of flexure of elastic materials. In case of a simple beam, with a uniformly distributed total load, W, a center deflection, f, and a span, L,

$$E_c = \frac{5\,W\,L^3}{384\,f\,I} \qquad (72)$$

The moment of inertia, I, is based on the transformed-section, which first requires an assumed value, n. With E_c obtained from equation (72), a corrected value, n, could then be substituted in equation (50). For approximate purposes it will be permissible to assume, $I = I_c$.

As an example, consider beam No. 46, where the load increase from 2000 to 42,000 lbs. gives a deflection, $f = .267$ in. With I, from Table 5, and $L = 276$ in., the instantaneous modulus of elasticity is,

$$E_c = \frac{5 \times 40{,}000 \times 276^3}{384 \times .267 \times 7182} = 5{,}720{,}000 \text{ psi.}$$

12—CONCLUSION

The design principles previously outlined can be applied, with few modifications, to prestressed members of various types. The most economical shape does not necessarily coincide with conventional sec-

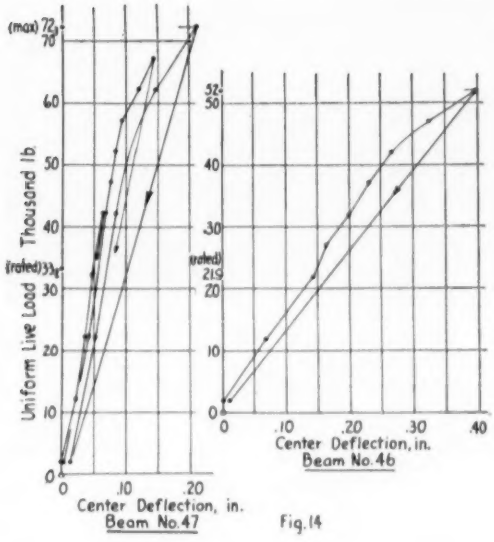


Fig. 14

Fig. 14

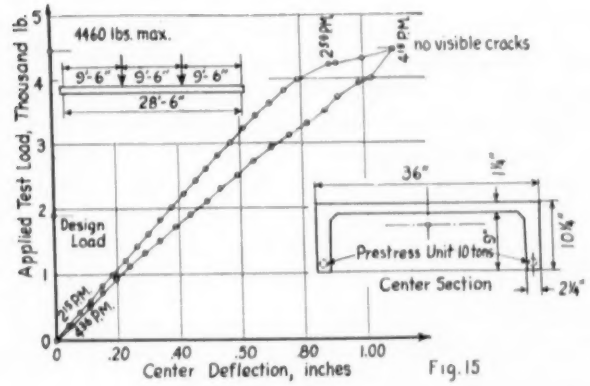


Fig. 15

Fig. 15

tions. It will be found that flanged shapes are advantageous in case of beams, girders, frames, etc., due to the high shear resistance of prestressed members.

Bridge structures of the monolithic deck and girder type require prestress action in two or more principal directions, with excellent resistance to the effect of concentrated wheel loads, impact and vibration. Long span bridges of the simple beam and cantilever type can be designed with a relatively small depth-span ratio, combined with small deflections. Prestressed ties and struts offer interesting possibilities in concrete truss construction.

Considerable steel reductions are gained already by prestress action in the principal direction of the span only. Fig. 16 shows a precast floor

Fig. 16—Precast floor construction, with beam sections similar to that shown in Fig. 15

construction, consisting of beam sections similar to Fig. 15, reinforced with prestressed units in the longitudinal ribs and ordinary wire mesh in the slab. The combination of prestressed reinforcement and ordinary reinforcing bars, acting in the same direction, should preferably be avoided. Such designs lead to similar difficulties, as found in ordinary reinforced concrete columns.

The application of monolithic prestress action to shell structures, such as round tanks, Zeiss-Dywidag roofs, etc., introduces other problems, which cannot be discussed here. In this connection it should be pointed out that the proposed method of construction is a comparatively new development. It will therefore be advisable to proceed from simple structures to more complicated applications.

The theoretical derivations and test results lead to the following tentative conclusions.

1—Cracks in ordinary reinforced concrete structures definitely limit the permissible steel stress and therefore also the working stress of concrete in compression, shear and torsion.

2—Prestressed reinforcement removes the principal objection to higher working stresses and permits the efficient utilization of high yield point steel wire in combination with high strength concrete.

3—A low steel stress produces a low and rather uncertain concrete prestress, obtained with a waste of steel.

4—A high steel stress is essential in order to obtain a high and reliable concrete prestress, combined with a minimum amount of steel.

5—The instantaneous deflections of prestressed beams indicate a nearly constant modulus of elasticity, at least over the entire range of compressive stresses. Upon removal of loads, the recovery of deflections is rapid and almost complete, even in case of hair cracks produced by excessive loads.

6—The shear resistance of prestressed beams and girders is remarkably high. It appears that the principal tensile stress, rather than the shearing stress, determines the critical load condition at the supports.

7—Prestressed concrete, compared to ordinary reinforced concrete, permits steel savings up to about 85 per cent and weight reductions up to about 70 per cent. The economies are most pronounced in case of heavy loads and long spans.

8—The cost of prestressed concrete depends largely on the amount of special equipment and the possibility of repeated use of the same. Self-contained, prestressed reinforcing units are proposed as a practical solution for cast in place structures.

Acknowledgment

The author is greatly indebted to Mr. E. J. Critzas, who has been in direct charge of all the experimental work done in St. Louis, Mo., since 1939.

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Discussion of this paper should be submitted in triplicate to the ACI Secretary before September 1, 1943, for publication in the JOURNAL Supplement concluding Proceedings V. 39.

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Discussion of a paper by Herman Schorer:

Prestressed Concrete, Design Principles and Reinforcing Units*

DISCUSSION BY DOUGLAS E. PARSONS, PAUL W. ABELES and the AUTHOR

By DOUGLAS E. PARSONS

A broadening of interest in prestressed reinforced concrete is evidenced by the appearance of three papers on the subject in a single issue of the JOURNAL. Crepps (p. 545) described methods of design and production and properties of pipe. Largely as the result of his work and that of J. E. Miller of the Lewistown Pipe Co., prestressed reinforced concrete pipe are no longer a laboratory novelty. Such pipe have demonstrated their reliability and usefulness. Billner and Carlson (p. 585) describe prestressing by a novel method involving the temporary heating of the reinforcement by passing an electric current through the bars after the concrete has hardened. They also demonstrated that the method is more than a mere idea.

Schorer discussed the nature of the deformations of a member during and subsequent to prestressing and gives a clear explanation of the factors that control them. Methods for estimating the magnitudes of the deformations and stresses in the concrete and reinforcement are described and formulas for the design of simple structural members are given. Original test data also are given. These as well as the design data are of general application, irrespective of the method of prestressing.

As pointed out by the author, the potential advantages of prestressed concrete were not realized in some early applications because of the use of low values of the original prestress. With low values, the residual stresses cannot be estimated accurately and the reinforcement is not used efficiently. With higher values, errors in estimating the reduction in stresses resulting from shrinkage and creep become negligibly small, the amount of steel reinforcement needed is less, cracking is prevented,

*ACI JOURNAL, June 1943; *Proceedings* V. 39, p. 493.
†Chief, Masonry Construction Section, National Bureau of Standards, Washington, D. C.

and the full area of the concrete is effective in resisting stresses. The problem of obtaining adequate bond strength is simplified by the use of wire or small bars, and savings in the weights of structures and of steel may be realized by the use of high-strength reinforcement. The practical advantage of the use of quick-hardening concrete also is pointed out.

Without advocating prestressing for all structures, the author indicates its usefulness in several types of structures. Beams and girders of very long spans, but of modest dimensions and weight, are not practicable if of ordinary reinforced concrete, but are feasible and economical if a simple and inexpensive method of prestressing can be used. The reduction of diagonal tension stresses by the prestressing of flexural members offers one solution to a troublesome problem that sometimes confronts designers. The prestressing of some types of precast products, such as piles, beams and slabs, appears promising as a possible means to minimize cracking. As indicated in the paper, the practical limit of design tensile stresses in reinforcement often is governed by the widths of cracks that may be tolerated. Consequently, design stresses in liquid-retaining structures usually are much less than for others, unless prestressed.

The ingenious self-containing prestressing unit offers a means of utilizing the prestressing principle when it would not be feasible to apply the needed forces by external devices. Although the full range of usefulness of the unit has not been ascertained, the data given in the paper and information gained from an examination of the units indicate that the magnitude of the stresses can be ascertained and controlled satisfactorily.

The simple equations (1) to (5), inclusive, expressing empirical relations between the compressive strength and other properties of concrete are useful for making rough estimates. Examinations of more comprehensive data, including those cited by the author, indicate that closer approximations are given by equations of similar form but with a higher exponent for S_c. For a set of equations in which the exponent is the same for all, a value of 0.7 appears to give the best fit. The corresponding equations are:

$$S_t = 1.1\ S_c^{0.7}, \quad S_f = 1.8\ S_c^{0.7} \quad \text{and} \quad E_c = 12{,}000\ S_c^{0.7}$$

The actual relations depend importantly upon many factors, including kind of aggregate and age, moisture content, and curing conditions of the concrete. Moreover, the substitution of the foregoing equations for those given by the author would not change the qualitative indications of his conclusions.

Equation (8) offers a means for estimating the effect of humidity during storage on the shrinkage of concrete. The effects of other factors on the magnitude of shrinkage often are difficult to estimate when design studies are being made. Uusually the structural designer will not have

complete information on the composition of the concrete, and sometimes may know only the specified strength and slump of the concrete and the maximum size of the aggregate. In such instances, and in the absence of more accurate means for making estimates, the following empirical equation may be of assistance:

$$e_s = \left[0.02 + \frac{0.045}{\sqrt{D}}\left(1 + \frac{8.5}{12 - S}\right)\right]\left(0.9 - h\right),$$

in which e_s = shrinkage, per cent
D = maximum size of aggregate, inches
S = slump, inches
h = relative humidity

More data are needed before methods of making reliable and close estimates of shrinkage and creep can be developed. In particular, the quantitative effect of size of member on the amounts of shrinkage and creep, while important, cannot be estimated with precision. It is believed that in most instances equations (9) and (10) will afford estimates sufficiently accurate for use in the design of prestressed members. The evidence available indicates, however, that equations of the forms studied by Thomas*, Ross†, Lorman‡ and McHenry** give a better indication of creep under long-time loads than those of the power type.

Fortunately, accurate estimates of shrinkage and creep are not needed for the successful use of the prestressing method if the original prestress is sufficiently large. In fact, in recent use of the method in Europe, less consideration of the effects of shrinkage and creep appeared to be given than in the design procedure outlined by the author.

By PAUL W. ABELES††

Mr. Schorer says in the introduction to his conclusions: "The combination of prestressed reinforcement and ordinary reinforcing bars, acting in the same direction, should preferably be avoided. Such designs lead to similar difficulties, as found in ordinary reinforced concrete columns."

The writer is not aware of difficulties found in ordinary reinforced concrete columns and should like to know on what facts Mr. Schorer has based his statement. Mr. Schorer has related his considerations to a design in which a stretching force of such a magnitude is applied that tensile stresses are avoided under working load conditions. This process which may be called "full" prestressing was proposed by R. E. Dill of

*A Conception of the Creep of Unreinforced Concrete, and an Estimation of the Limiting Values, *Structural Engineer*, V. 11, p. 69 (1933).
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††Technical Advisor, London.

Alexandria, Nebr., in 1923* for destroyed bond between reinforcement and concrete and later by the French engineer, E. Freyssinet in 1928† for bonded reinforcement. Both these inventions intended to permit high strength steel to be used in reinforced concrete. Since no tensile stresses occur under working load there is no need to limit the working stresses, which limitation is done in conventional practice only for the purpose of avoiding dangerous cracks. With the use of high strength steel it was to be expected that dangerous cracks would develop in a prestressed structure, when total absence of cracks is not ensured,—in view of the high steel stresses appearing in a cracked section. This limitation to "full" prestressing is, however, only necessary if the bond of the high strength steel reinforcement is destroyed according to the proposals of Dill and Hewett, but not necessary when the whole reinforcement or at least a part of it is bonded to the concrete. On the basis of comparative tests on beams reinforced with high tensile wire it is seen that "partly" prestressed concrete structures behave when under working load similar to concrete structures reinforced with mild steel in conventional practice, dangerous cracks being avoided, but that they behave like homogeneous constructions when the working load is removed.

In fact, the use of prestressed reinforcing units according to Schorer for ¼ to ½ of the total high strength steel reinforcement (instead of for the total tensile reinforcement) and the replacement of the rest by unstretched high strength wire would result in improved economy. A homogeneous material with guaranteed absence of cracks would not be obtained (such material would be required only for special purposes), but the properties of the structures would be the same, when under working load, as in a conventionally designed structure; they would even be considerably improved in comparison with those in usual construction, when the working load is removed, although the tensile reinforcement is reduced to about 1/5 of that in usual design. However, the shear resistance is smaller in partly than in fully prestressed structures, thus necessitating a stronger shear reinforcement.

Reference may be made to beams 48 and 49 in which top cracks occurred when the compression was transferred to the concrete. This proves that it is advisable to provide also a prestressed top reinforcement where it is intended to obtain total absence of cracks. The writer has derived formulas for this case in addition to an equation for the initial stretching force to be applied in order to ensure that no tensile stresses occur under working load even if the greatest possible losses of the initial prestress are considered.

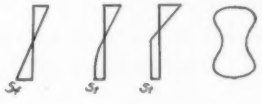
*U. S. Pat. No. 1,684,663—(See R. E. Dill, "Some Experience with Prestressed Steel in Small Concrete Units," ACI JOURNAL, Nov. 1941; *Proceedings* V. 38, p. 165—EDITOR).

†U. S. Patent No. 2,080,254.

It ought to be borne in mind that at failure wide cracks have already developed. The design of the tensile reinforcement, required to ensure the requisite factor of safety against breaking, has therefore to be based on a cracked section as in conventional practice. Thus it appears advisable to reserve the conventional signs f_c and f_s for the cracked section, and clearly to differentiate, by other signs, the prestress in the steel from the concrete stresses occurring in the outside fibres of the sections. From the expression "S_c = compressive strength of concrete" it is not seen which strength Mr. Schorer has in mind. The cylinder strength is known by the symbol f'_c; there is further a cube strength and a prism strength. If it is the cylinder strength it appears to be advisable to keep the standard notation f'_c and also to replace S_f and S_t by other symbols corresponding to f'_c.

Under the headline "Cracks" Mr. Schorer says: "The flexural strength S_f is more easily determined by tests" (than the tensile strength S_t) and later "Cracks develop when the concrete stress exceeds the approximate limits given by equations (1), (2), (4) and (5)." The writer cannot agree with these statements. First the difference between flexural strength S_f and tensile strength S_t may be discussed. The flexural strength, also called "tensile bending strength" or "Modulus of rupture," is only an arbitrary value and is not measureable; it is computed for an assumed triangular straight line stress distribution (Fig. A). But in practice, at

Fig. A, B, C, D left to right



rupture the stress distribution, in the tensile zone of an unreinforced concrete section, or at cracking in that of a reinforced concrete section, is either curved (Fig. B) or rectangular (Fig. C) due to the plasticity of the material, the greatest stress being equal to the tensile strength of the concrete. Although the Modulus of rupture varies greatly for different dimensions (shapes and sizes of section) and plasticities even for the same tensile strength, the latter is "not appreciably affected by either the length or size" of the specimens according to tests conducted by Gonnerman and Shuman* The Modulus of rupture is in accordance with various test results 1.5 to 2 times the tensile strength. It is agreed that it is generally easier to make bending tests than pulling tests on rather complicated specimens. However, the results of bending tests differ greatly, as pointed out, depending on the dimensions of the specimens and on the arrangement of the loading. Such tests are therefore of little use. On the other hand, tests on briquettes of a section 1.5 in. by 1.5 in. as used

*"Compression, Flexure and Tension Tests of Plain Concrete," by H. F. Gonnerman and E. C. Shuman, *Proc.* ASTM, V. 28, 1928.

for the testing of cement (Fig. D) have proved very simple and give satisfactory results. The writer advocates this method which does not require much material and allows of a very simple testing, if clamping devices are at hand. (The size could advantageously be increased to, say, 2 in. by 2 in.).

Independent of these objections to details in Mr. Schorer's paper, the writer suggests that the prestressed reinforcing units suggested by Mr. Schorer represent a great improvement in the method of prestressing, since they allow the stretching process to take place in a factory (which should result in economy due to the application of factory methods) and permit the molds to be removed in the usual manner, without it being necessary to wait until the concrete has attained sufficient strength.

AUTHOR'S CLOSURE

Mr. Parsons' discussion is gratifying. His suggestion of more accurate expressions, in place of equations (1), (2) and (8), is a desirable improvement.

The forms of creep equations cited by Mr. Parsons are undoubtedly better adapted for the expression of long-time load effects. The types suggested by Ross and Lorman appear preferable, for reasons of simplicity. However, as shown by Shank[15], the power type gives good conformity during the early flow period. The two forms may be combined in a more general expression, so as to obtain good conformity over the entire flow period. With B, C and q as constants, the following form is suggested,

$$y = \frac{B\left(\frac{x}{\alpha}\right)^q}{C + \left(\frac{x}{\alpha}\right)^q} \quad \ldots\ldots (73)$$

It will be noted that the power function dominates during the early flow period. With increasing values, x/α, the flow approaches the so-called limiting value, B.

Equation (10) may be stated in the improved form,

$$y = \frac{1.50}{1{,}000{,}000}(1.25 - h)\frac{\left(\frac{x}{\alpha}\right)^{2/3}}{1.50 + \left(\frac{x}{\alpha}\right)^{2/3}} \quad \ldots\ldots (74)$$

With $x/\alpha = \infty$, instead of equation (11),

$$y = B = \frac{1.50}{1{,}000{,}000}(1.25 - h) \quad \ldots\ldots (75)$$

Mr. Parsons points out that recent methods used in Europe give less consideration to the effects of shrinkage and creep in the design of prestressed members. For instance, the tentative German code, intended for beams and slabs of the Hoyer type, simply specifies a constant deduction of about 21,000 psi from the original steel prestress. This procedure appears justified in view of the high steel prestress (175,000 to 250,000 psi), a condition expressed by Mr. Parsons.

Mr. Abeles finds certain advantages in "partly" prestressed structures. The width of cracks under normal loads can be reduced. However, the unstretched reinforcement will in time be subjected to considerable compressive stresses, which require closely spaced stirrups, lateral ties or spirals, as in columns, in order to counteract possible buckling tendencies of the longitudinal bars, as reported by Mörsch[6]. Mr. Abeles' claim for improved economy appears questionable, since the comparison cannot be made on the basis of equal quality.

The expression, S_c, represents the cylinder strength, which should have been designated as suggested by Mr. Abeles.

The apparent difference between flexural and tensile strength has been explained in detail by Mörsch[6] and others. The assumption of a straight line stress distribution in prestressed members has generally been accepted for practical design purposes. The consideration of crack loads requires due modifications.

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The Function of Entrained Air in Concrete*

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Member American Concrete Institute

SYNOPSIS

The use of air entraining agents in portland cement concrete shows considerable promise, particularly in highway concrete where resistance to scaling is an important factor. The need for fundamental research regarding the mechanism by which entrained air functions is suggested and methods of tests are outlined.

Some results of exploratory investigations of the action of air entraining agents on the individual constituents of concrete are reported. It is indicated that the action of air entraining agents in cement paste differs materially from their action in concrete. The effect of air entraining agents on neat cement paste suggests a probable maximum cement content of concrete, above which the beneficial effect of air entraining agents may become negligible, their effect on strength remaining unchanged.

During the years 1932-1935, intensive work was done by the Dewey & Almy Chemical Co. in connection with the application of a combined air entraining and dispersing agent to portland cement concrete. Cooperative studies made by Prof. C. H. Scholer in the early thirties showed that the entrainment of small amounts of air improved the durability of such concrete. Parallel studies were made by this company which included investigations of the effect of air entraining agents on other properties of concrete, also the individual effects of (1) the air entraining agent, and (2) the entrained air. Though the beneficial effects of air entraining agents on resistance to freezing and thawing were recognized, the accompanying reductions in density and strength were deemed sufficient reason to discontinue work on this subject at the time.

But in recent years, interest in the possibility of increasing concrete durability by means of air entrainment has been revived, largely because of the scaling effects of $CaCl_2$ used to de-ice pavement surfaces. A review

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of these earlier results interpreted in the light of present day knowledge and experience may be of interest to concrete engineers and cement technicians with the presentation of findings of more recent research.

It was concluded early in our first work that plasticity or workability of concrete—always a prime consideration—could be markedly improved by the use of two general classes of additions, (1) catalyst-dispersing agents which function by lubricating the cement without entraining appreciable air, thus increasing the fluidity of the cement paste matrix in concrete, and (2) many air entraining agents, such as some wetting agents, resins, etc., introducing well distributed air bubbles which appear to act as ball bearings and decrease the particle interference of the concrete aggregates. This paper concerns only the second class.

A full appreciation of the function of entrained air is necessary for a thorough understanding of the probable effect of air entraining agents on the resulting hardened concrete. To date many data have been accumulated on the ultimate effects of air entraining agents on concrete. However, little or no work regarding the mechanism by which air entraining agents achieve these effects has come to the attention of the author. It is therefore the purpose here to supplement and not duplicate available data and to point out a new line of investigation which shows considerable promise as a means of ascertaining the actual function of entrained air.

The following discussion assumes that air entraining agents are added in amounts sufficient to produce 3-4 per cent added entrained air, from .01 to .05 per cent of such agents by weight of the cement generally being found suitable.

EFFECT OF AIR ENTRAINING AGENTS ON CONSISTENCY

Freshly mixed cement paste

If a paste is made with a cement containing certain wetting agents or resinous air entraining agents using the normal amount of agitation or mixing, much less air may be entrained than is entrained in a comparable concrete. (The term "entrained air" when used in this paper, designates the air entrained in addition to that normally entrained in the absence of agents.) Despite this air entrainment, no increase in plasticity of the cement paste can be discerned either by visual inspection or by any of the generally used methods of determining plasticity or mobility. In fact it has generally been noted that a slight stiffening in the paste occurs when air entraining agents are incorporated in the paste, due undoubtedly to additional immobilization of the gauging water by the entrained air.

It should be noted that a larger amount of air can be entrained in cement pastes containing air entraining agents if the mixing is abnormally vigorous. The amount of air entrained in the treated cement paste

can be doubled by increasing the time or vigor of mixing. The same is true of concrete. A fair comparison of air entrainments in cement paste and in concrete can be made only when the mixing is equivalent in time and vigor.

The action of air entraining agents on neat cement may be illustrated as follows.

Add 60 grams of cement to each of two 150 cc beakers. To one beaker add an amount of water necessary to provide a consistency approximating that of heavy cream (approximately 28 cc) for normal cement. To the second beaker add the same amount of water and incorporate the recommended amount of air entraining agent. After mixing the contents of the two beakers thoroughly by means of a spatula, it will be noted that there is little or no difference in the consistency of the two pastes.

Mixtures of fine aggregate and water

In spite of these results, it is well established that air entrained in concrete through the medium of an air entraining agent, imparts additional workability to that concrete. Hence it follows that the additional workability imparted to concrete through the medium of an air entraining agent must be due directly to the action of this air in decreasing particle interference of the concrete aggregates. This action is demonstrated as follows.

Place 25 grams of sand, for instance standard Ottawa sand, in each of three 150 cc beakers. Mix thoroughly 12 cc of water with the sand in one beaker. A harsh mixture readily adhering to the side walls of the beaker will result. To the sand in the second beaker add the same amount of water, this time including an air entraining agent. If, after mixing thoroughly, the entrained air is removed by applying a vacuum, this mixture will be harsh like that in the first beaker, despite the presence of an air entraining agent. In the third beaker, add the same amount of water with the air entraining agent but omit the vacuum. This mixture will become realtively plastic and even fluid. This phenomenon is attributable to the entrained air which apparently acts as small ball bearings and lubricates the aggregate. These simple tests are illustrated in Fig. 1.

Freshly mixed concrete

If as indicated in the foregoing, air entraining agents act principally on the sand or fine aggregate constituent of a concrete, it follows that lean mixes will entrain correspondingly more air than rich mixes. Hence lean mixes will exhibit greater improvement in workability or consistency, thus permitting appreciable reductions in water-cement ratio for a given slump.

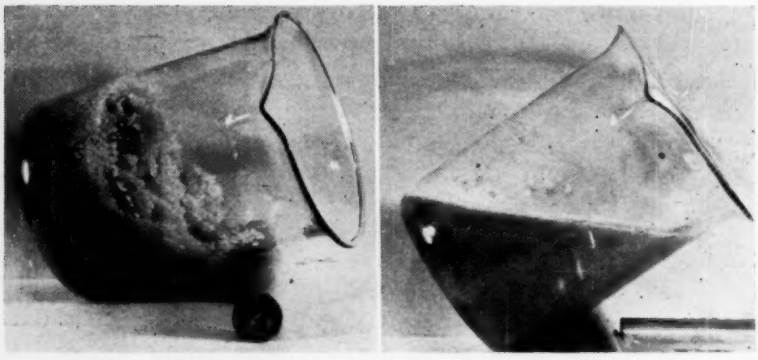


Fig. 1—Effect of air entrainment on flowability of sand-water mixtures. Left, no entrained air; right, with entrained air; same water content in both cases

Rich mixes which have relatively less of the fine aggregate constituent, will show less entrained air, the amount progressively diminishing with increasing cement content. It is reasonable to believe that a maximum cement content will be reached beyond which the percentage of air is so reduced that no improvement in consistency is obtained, despite the presence of the air entraining agent. In this case no reduction in water-cement ratio can be made.

EFFECT OF AIR ENTRAINMENT AGENTS ON DURABILITY

Investigation of hardened cement pastes

It has been found by many investigators that the resistance of concrete to freezing and thawing is definitely improved by the use of air entraining agents. However, work in this laboratory and by other investigators, has shown that little or no improvement in resistance to freezing and thawing may be obtained with neat cement pastes and concrete of high cement content. For example M. A. Swayze in his article, "More Durable Concrete with Treated Cement" shows no appreciable improvement due either to resin or tallow at a cement content of 7.5 bags per cu. yd. and his data indicate by interpolation that there is a critical cement content of 6.5 to 7.0 sacks per yd. above which negligible improvement in durability can be obtained by employing air entraining agents.* Durability studies of neat cement pastes which are interesting because they represent the limit of increasing cement content, were conducted in this laboratory.

Four sets of neat cement prisms 1.5 x 1.5 x 6 in. were made using an identical water-cement ratio of 0.27 by weight. Two sets were made with a Type II cement clinker ground to a surface area of 1895 cu. cm. per gm.,

**Engineering News-Record*, June 19, 1941, V. 126, p. 946.

and the other two were made with the same clinker ground with .03 per cent of a resinous air entraining agent to a surface area of 1880 cu. per cm. gm. In each case two types of mixing were used, hand mixing and with a high speed mechanical mixer. Thus the air entrainment was varied by the method of mixing as well as by the incorporation of resin.

All specimens were standard cured for seven days, the short period being selected because tests on concrete containing the same cement and air entraining agent had shown less decrease in strength at 7 days than at the later ages. At the end of this period, the surface dry weights and density figures were obtained for each specimen. The specimens were then repeatedly frozen and thawed in a 10 per cent solution of calcium chloride. All specimens were frozen at −15 F for 24 hours and then thawed at 70 F for 24 hours. Hence one complete cycle was obtained every two days.

Fig. 2 shows the condition of the specimens at the end of 21 cycles. It may be seen that specimens containing the air entraining agent scaled more than the corresponding blank specimens. It is also apparent that the high speed mixing had no effect, despite greater air entrainment.

Fig. 3 shows clearly that at the end of 105 cycles, the disintegration of the specimens containing the air entraining agent had progressed to a very marked degree. On the other hand, the untreated cement specimens showed only slightly more disintegration than at 21 cycles. In both cases the cores of the specimens seemed to be dense and sound.

It should be emphasized that these results were obtained on one cement only and at the one curing period and hence general conclusions cannot yet be drawn from these data. Tests with this same air entraining agent in two other cements are now in progress and show that the rate of disintegration after 50 cycles is much less pronounced.

Durability of highway concrete

There seems little question but that highway concretes with cement contents of 5-6 sacks per cu. yd. containing air entraining agents, have shown markedly improved resistance to freezing and thawing and little or no tendency to scale upon the application of calcium chloride for de-icing, despite a decrease in density and strength of the resulting concrete. Most authorities agree that the reduction in compressive strength is not serious since sufficient strength remains to carry whatever loads are imposed upon the highway. The decrease in bond strength is of little importance because bond is not an important factor in the design of concrete highways. Elimination of the tendency of the concrete to scale is a decided advantage.

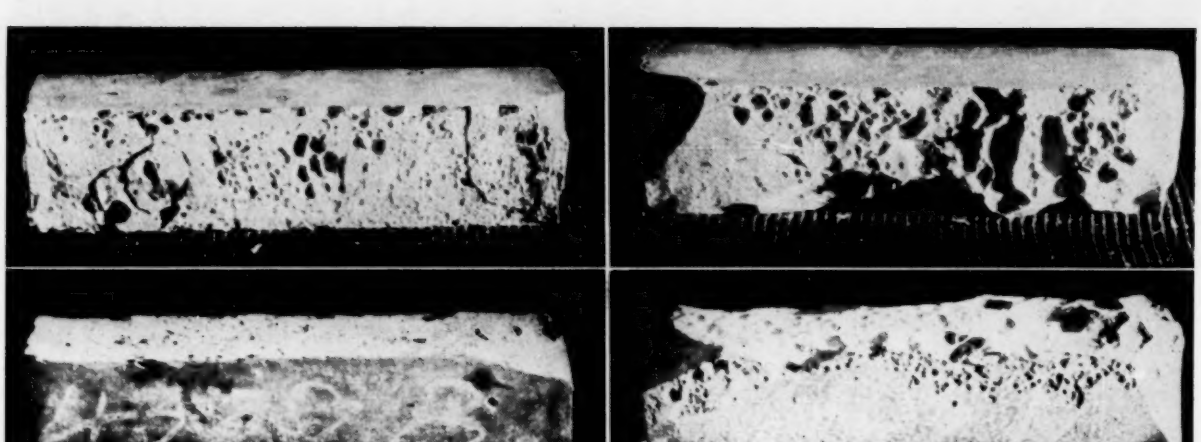


Fig. 2—Neat cement specimens after freezing and thawing for 21 cycles in 10 per cent $CaCl_2$ solution

Top left—cement paste without air entraining agent, density 2.22 mixed at slow speed. (top facing)

Bottom left—cement paste without air entraining agent, density 2.25, mixed at high speed. (top up)

Top right—cement paste with air entraining agent, density 2.14, mixed at slow speed. (top facing)

Bottom right—cement paste with air entraining agent, density 2.05, mixed at high speed (top up)

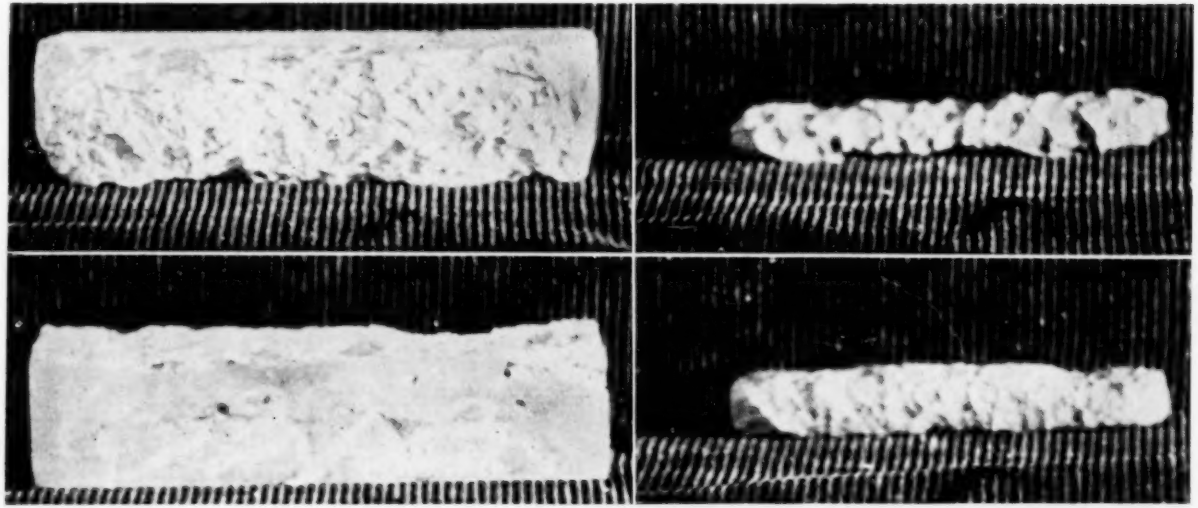


Fig. 3—Neat cement specimens after freezing and thawing for 105 cycles in 10 per cent $CaCl_2$ solution

Top left—cement paste without air entraining agent, density 2.22, mixed at slow speed. (top facing)

Top right—cement paste with air entraining agent, density 2.14, mixed at slow speed

Bottom left—cement paste without air entraining agent, density 2.25, mixed at high speed. (top up)

Bottom right—cement paste with air entraining agent, density 2.05, mixed at high speed

It is generally accepted today that the minimum additional air required to increase materially the resistance to freezing and thawing is 3 per cent of the volume of the concrete. In many cases the entraining of still larger percentages of air with a consequent greater decrease in density will still further increase resistance to freezing and thawing, but this added improvement is of doubtful value in view of the lower strengths and densities. Hence it seems desirable to keep the air content as low as possible, and to keep the strengths and density as high as possible, while achieving the desired results.

The effect of entrained air on resistance to freezing and thawing seems to be independent of the means employed to entrain the added air. One of many different air entraining agents including fatty acids, wetting agents, oils and resins can be used. The beneficial effect of the air entrained by such agents on freezing and thawing is general, but the decreases in strength and density of concrete vary considerably with the type and amount of the agent used. If it is true that similar improvement in durability can be obtained from several different agents, the basis for acceptance of any such agent may be its effect on strength when it is used in amounts to entrain approximately 3 per cent of air in the concrete. If proper dispersion of the air through the concrete is obtained, air entrainment is likely to be beneficial regardless of how it is derived. Such gas generating agents as aluminum powder and hydrogen peroxide have been used with good result.

The foregoing suggests that the entrained air and not the air entraining agent is responsible for the improved resistance to freezing and thawing obtained. That this is so is substantiated by the fact that concretes of high cement content in which air entraining agents are incorporated and which contain very little entrained air, show little or no improvement in resistance to freezing and thawing. In this connection, it is interesting to note the results of tests made on normal concrete mixes (cement content 6 sacks per cu. yd.) containing a common resinous air entraining agent.

To a portion of each freshly mixed concrete used, a vacuum was applied in such a way that only the extra entrained air and no mixing water were removed. Specimens were made both with and without the extra air removed for freezing and thawing in a 10 per cent solution of calcium chloride. Control specimens of the same cement content without the air entraining agent were made for comparison.

Typical data based upon comparative mixes of (1) identical w/c and (2) identical slump are shown in Fig. 4 and 5. The concrete containing the added entrained air, as indicated by the densities tabulated, show markedly improved resistance to chloride action whereas the concrete

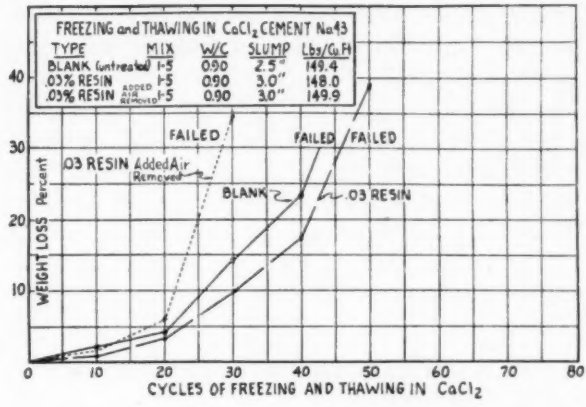
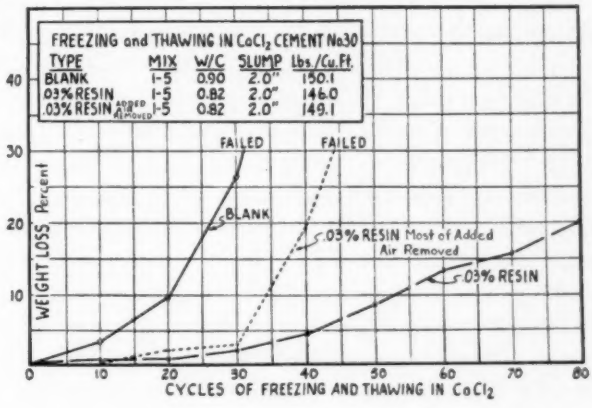

FREEZING and THAWING IN $CaCl_2$ CEMENT No.43
TYPE MIX W/C SLUMP Lbs/Cu.Ft.
BLANK (untreated) I-5 0.90 2.5" 149.4
.03% RESIN I-5 0.90 3.0" 148.0
.03% RESIN ADDED AIR REMOVED I-5 0.90 3.0" 149.9
FAILED
FAILED
FAILED
.03 RESIN Added Air Removed
BLANK
.03 RESIN
WEIGHT LOSS Percent
CYCLES OF FREEZING AND THAWING IN $CaCl_2$

Fig. 4—Freezing and thawing in $CaCl_2$—cement No. 43

Fig. 5—Freezing and thawing in $CaCl_2$—cement No. 30


FREEZING and THAWING IN $CaCl_2$ CEMENT No.30
TYPE MIX W/C SLUMP Lbs/Cu.Ft.
BLANK I-5 0.90 2.0" 150.1
.03% RESIN I-5 0.82 2.0" 146.0
.03% RESIN ADDED AIR REMOVED I-5 0.82 2.0" 149.1
FAILED
FAILED
BLANK
.03% RESIN Most of Added Air Removed
.03% RESIN
WEIGHT LOSS Percent
CYCLES OF FREEZING AND THAWING IN $CaCl_2$

containing the same air entraining agent but from which the air has been removed by evacuation, shows little or no improvement. In fact, in Fig. 4 from which more than the added entrained air was removed, the concrete shows decreased durability. Note that specimens containing no air entraining agents are designated as "blank."

Air entraining agents in structural concrete

Structural concrete, unlike highway concrete, is seldom if ever exposed to the disintegrating effect of $CaCl_2$. It is however, very frequently exposed to alternate cycles of freezing and thawing in the presence of excess moisture. Hence for clarity of thought on this subject, a sharp line of distinction should be drawn between durability as measured by freezing and thawing in the presence of chlorides and durability as measured by freezing and thawing in plain water.

For example, it has been found that in general, for any given concrete mix, resistance to cycles of freezing and thawing in water increases with

increasing fineness of the cement.* The reverse of this may be true, however, for freezing and thawing in the presence of chloride solutions.

Entrained air will improve the resistance of concrete to freezing and thawing in both water and chloride solutions. However, in water, the usual criteria of density and strength apply. Hence, equivalent durability is obtainable under such conditions simply by increasing the cement content which will at the same time result in greater strengths. Since strength is a primary consideration in structural concrete, the use of air entraining agents with their attendant reductions in strength are of questionable value as a means of increasing resistance to straight freezing and thawing.

EFFECT OF AIR ENTRAINING AGENTS ON STRENGTH OF HARDENED CONCRETE

It is obvious that the entrainment of air will result in decreased density of concrete. It follows logically that this decrease in density has a corresponding effect on strength—tensile, compressive, flexural and bond. In addition some air entraining agents may in themselves be detrimental to strength of concrete so that the total reduction in strength is greater than that due to air entrainment alone. This conclusion is indicated by the fact that with certain air entraining agents, rich mixes which contain less added entrained air than lean mixes show about the same decreases in strength as lean mixes.†

There is an explanation for the large decrease in strength obtained at identical water ratio in that the bulking of the mortar through the entrainment of air has an effect similar to that obtained when additional water is added to the mix to produce the same consistency. When a water cut is taken so that the concretes have comparable slumps, some of this bulking effect is compensated with the result that part of the reduction in strength is eliminated.

The average decrease in strength for the commonly known air entraining agents when used in amounts to cause 3-4 per cent increase in air content, is perhaps 20-25 per cent. This 20-25 per cent figure however, holds only when the water has been reduced so that the concrete will have a consistency comparable to that of a concrete made in the absence of the air entraining agent. At identical water-cement ratio, the presence of an air entraining agent in the same amount may cause decreases in strength of as much as 40-60 per cent.

The bulking of concrete due to entrained air results in a lower cement content per unit volume of hardened concrete than would be theoretically calculated from the absolute volumes of the individual constituents.

*A. G. Timms: "Effect of Duration of Moist Curing on the Principal Properties of Concrete," Bulletin, Portland Cement Association Research Laboratory.

†M. A. Swayze, *Engineering News-Record*, June 19, 1941; V. 126, p. 946.

If the theoretical cement factor is increased to compensate for this bulking, thus producing the same cement factor in hardened concrete as obtained with the untreated cement, some of the loss in strength will be counteracted. However, it should be borne in mind that the concrete having the greater theoretical cement factor will contain less entrained air. Therefore, since the percentage of entrained air is a decided factor in improving durability as measured by freezing and thawing, less improvement in durability may follow.

The bulking of concrete due to air entrainment naturally results in decreased settlement of the cement paste and aggregate with a consequent reduction in total bleeding and water gain under the aggregate. It is reasonable to expect therefore that by eliminating water gain under horizontal reinforcing steel, the bond between the steel and the concrete will be improved. This, however, is questionable since many investigators have found that in general, decreases in bond strength approximating the decreases in compressive strength cited above are obtained.

A possible explanation for this decrease in bond is indicated by Fig. 6 In the first case, with no air entraining agent present, rather severe channeling due to water gain is evident. But where an air entraining agent was involved, air instead of water appears to have been trapped beneath the reinforcing rod producing a highly porous mortar in contact with the bottom surface of the rod. The bond strength for the specimen with no entraining agent (top Fig. 6) was 201 psi at a slip of .0021 in. The bond strength for the lower specimen was 159 psi. at a slip of .0008 in., a decrease of approximately 20 per cent, due to the entrained air.

It must be pointed out that no definite conclusions regarding bond strength can be drawn at this time. A comprehensive series of bond tests reported by Wuerpel* shows no decrease in bond when an air entraining agent was present. It is worthy of note that in this case companion concrete specimens showed a 20 per cent increase in compressive strength.

SUMMARY AND DISCUSSION

Neat cement is not nearly as resistant to freezing and thawing as good concrete, despite the lower water cement ratio and higher strength. But lean concrete is not nearly as resistant as rich concrete; an optimum cement content seems to exist which is better than either very rich or very lean concrete.

When air entraining agents are used in concrete mixes having cement contents up to 6 sacks per cu. yd., the consequent small air voids distributed through the concrete cause a great improvement in resistance to freezing and thawing but similar air voids in neat cement appear to have

*C. E. Wuerpel: Cement Durability Program, First Interim Report, Central Concrete Laboratory, U. S. Engineers, 1942.

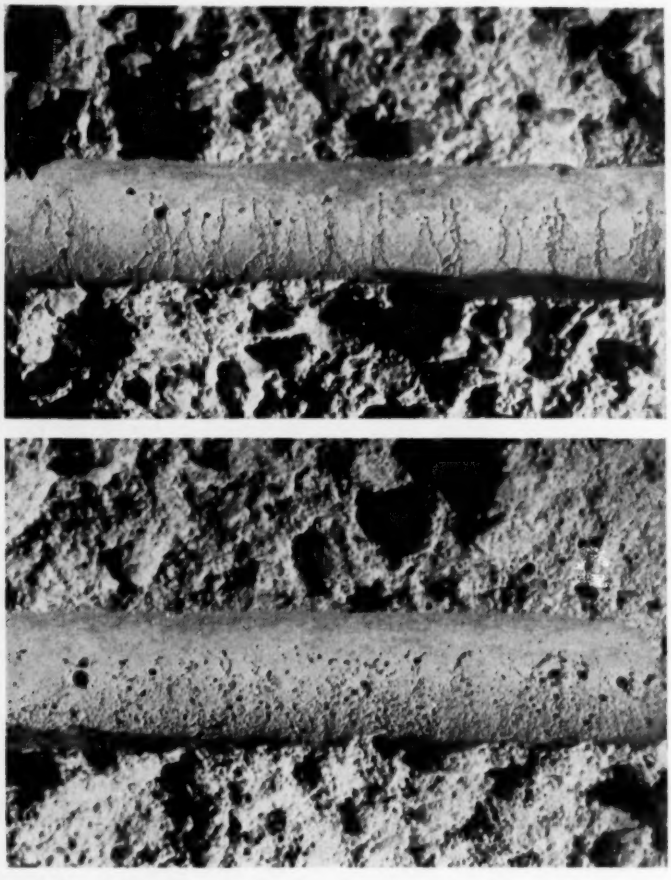


Fig. 6 — Bond Specimens. Appearance of concrete in contact with under side of ¾-in. steel rod (rod removed). Same cement and mix in each specimen.

(above)—No air entraining agent used.

(below)—Air entraining agent incorporated.

little or no beneficial effect. When frothing agents are used but the extra air is removed, no greater durability is found, even in concrete.

In neat cement, the low durability seems to be due to the higher volume change. In this connection, it should be noted that air voids appear to have little effect on volume change of neat cement as long as the water cement ratio is not altered. It has been shown by Carlson that while concrete has low drying shrinkage because of the restraint of the aggregate, the shrinkage of concrete approximates that of the corresponding neat cement when a highly compressible aggregate is used.* Thus, neat cement with air voids should shrink like neat cement without air voids. Of course, the air voids must cause some differences, such as affecting the rate of moisture movement, and reducing the modulus of elasticity.

*R. W. Carlson: Chemistry and Physics of Concrete Shrinkage; Proc. A.S.T.M., V. 35, Part II, p. 370, 1935.

But these are believed to be minor factors in neat cement, where restraint of aggregate is not involved.

It is well known that neat cement expands and contracts excessively due to moisture changes, and even its thermal expansion and contraction are much higher than those of common aggregates. It seems likely, therefore, that the shock of the freezing and thawing treatment might cause excessive stresses to be developed at the surfaces of a neat cement specimen due to the freezing and thawing treatment. These stresses appear to cause progressive scaling inward. If this explanation is correct, it would be expected that the relative durability of neat cements of equal water content should be as the relative strengths. The net result is that an air entraining agent which reduces strength of neat cement paste should be expected to reduce durability, as was found in the tests reported here.

In good concrete, the cement paste is so well diluted with the sounder aggregate that volume changes during the freezing and thawing treatment could hardly be expected to cause stress failure. It seems more likely that freezing and thawing failure is closely related to the size and distribution of the voids which are already in the concrete. This would explain why leaner concretes with larger and poorly distributed voids and lower strength are less durable. Also, it follows that aerated concretes which are more plastic and have well distributed small voids, should be more durable. However, the actual improvement in durability is greater than could be accounted for on this basis. The major effect of the entrained air in concrete is believed to lie in providing relief for the freezing expansion.

When a body of wet concrete freezes, the surface freezes first, thus sealing off the interior. This freezing of the surface film is not believed to cause any disruptive pressure because the outer film of water is not confined. As the freezing progresses inward, pressure tends to develop, but the pressure lowers the freezing point and the water creeps inward, as long as there are void spaces available, and the freezing progresses under little or no pressure. But as soon as the water has no ready pores to move into, the freezing causes disruptive expansion.

A remarkable feature of concretes containing well distributed air voids, is that the interior voids remain air filled and water free, even after long continued soaking. The interior air simply cannot escape. Thus when freezing progresses inward from the saturated surface, the water creeps inward and as soon as the saturated zone is passed, the danger of disruptive expansion is gone. In fact, it appears likely that the disintegration of concretes with a few per cent of well-distributed air may be due almost entirely to causes other than disruptive expansion of freezing water. There are always alternations of stress when concrete is subjected to

alternations in temperature and moisture, and these may be more serious than the freezing in aerated concretes.

CONCLUSIONS

The principal conclusion to be drawn from the limited data presented in this paper is that more work should be done along the lines here suggested. Further research of this nature will probably go far to provide a fundamental explanation of the action of air entraining agents in concrete and, perhaps more important, to ascertain the limitations of air entraining agents when applied to concrete in practice. That such limitations exist and should be given consideration seems evident from the information at hand.

Due consideration being given to the limited scope of the data so far available, certain other tentative conclusions may be drawn as follows:

1. Based upon comparable time and method of mixing and when using .01-.05 per cent of the air entraining agents discussed in this paper, (a) The consistency of the cement paste is not improved, (b) The consistency of fine aggregate and water mixtures is greatly improved.
Hence any improvement in the consistency of concrete due to entrained air must be due to the action of the air entraining agent on the aggregate alone.

2. The improvement in the durability of concrete seems to be due to the entrained air and not to the air entraining agent per se. When added entrained air is removed from freshly mixed concrete, all improvement in the durability of the resulting concrete is lost.

3. The failure of an air entraining agent to improve the resistance of neat cement paste to cycles of freezing and thawing indicates a critical limit in the cement content, above which little or no improvement in consistency and resistance to freezing and thawing of concrete can be expected.

Discussion of this paper should be submitted in triplicate to the ACI Secretary before September 1, 1943, for publication in the JOURNAL Supplement concluding Proceedings V. 39.

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Discussion of a paper by Henry L. Kennedy:

The Function of Entrained Air in Concrete*

DISCUSSION BY M. SPINDEL†

The paper gives me an opportunity to add my remarks in a previous discussion‡. Before I do so I would like to refer to some of my previous remarks about the oscillation of the opinion with regard to the amount of water in concrete as follows:

1. At the beginning the experts decided that the concrete had to be made as dry as possible and that it had to be consolidated by very much tamping until "sweating" set in. If somebody really insisted on the then standardized sweating, he often could see in a sweating state both the contractor and the laborers but not the concrete, though many stones had been broken by intense tamping. This type of concrete had not enough water and consequently too many very large air voids and did not resist to frost action.

2. As a reaction to the forementioned results and owing to the further development of reinforced concrete and the machinery for making mass concrete the pendulum swung excessively in the opposite direction. The concrete was made with an excess of water, the advantages of which had been highly praised. There was no tamping and no sweating and sometimes the concrete during placing looked like a swimming pool but the excess of water was disastrous even when not so clearly visible. This type of concrete had too much water and consequently far too many detrimental air voids after hardening, though of smaller sizes than those of concrete made too dry, and it did not resist frost action either.

3. The experience in the laboratory and in the field led me early to the conclusion that if a concrete mix was either too dry or too wet it was very hard to secure impermeability and resistance to frost etc. and that the concrete mixes had to be made with sufficient cement, well graded

*ACI JOURNAL, June 1943; *Proceedings* V. 39, p. 529.

†Member, Association of Austrian Engineers, Chemists, and Scientific Workers in Great Britain; London.

‡Mr. Spindel discussed in ACI JOURNAL, June 1943, p. 124-3, the paper by W. C. Hansen, "Influence of Sands, Cements and Manipulation upon the Resistance of Concrete to Freezing and Thawing," ACI JOURNAL, Nov. 1942; *Proceedings* V. 39, p. 105.

aggregates and only with as much water as was absolutely necessary to obtain a so-called stiff plastic consistency and good workability for being consolidated by slight tamping only. The concretes designed in this way for various kinds of structures including water tanks, large dams etc. proved to be watertight and resistant to frost action even under high water pressure and severe weather conditions. I must not omit mention that the above successes were due partly also to the use of admixtures, the advantages of which I have stated again and again without doubt. This type of concrete had just the permissible minimum of water and consequently just the still permissible amount of air voids very favorable in size, shape and distribution and it resisted frost action perfectly.

As far as high grade concrete for structural work is concerned we observe now similar oscillations of the opinion with regard to the amount of air voids in freshly made concrete as follows:

1. Some experts started to secure a minimum of air voids by applying a vacuum. I agree that the effect with regard to resistance to frost was rather disadvantageous. In my opinion that was due mostly to the fact that the action of sucking in vacuum had a detrimental effect on the size, shape and distribution of the pores still left, the latter particulars being much more decisive than small differences in the percentage of the pores.

2. As a reaction to the above experience and for other reasons the pendulum is just swinging excessively into the opposite direction, i.e. to a very high percentage of air voids by using special air entrainment agents. This has been found advantageous in spite of the enormous loss in strength and some other disadvantages and uncertainties. In my opinion the latter should not be disregarded as it happened more than 20 years ago, when the advantages of a high percentage of water blinded some of us to the disadvantages of it. As to the uncertainties of air entrainment agents I refer for example to the very instructive tests made by M. A. Swayze with regard to the effect of beef tallow added to his cement D which decreased strength and durability as well. Further I refer to the very interesting tests made by Henry L. Kennedy, the net result of which was that an air entrainment agent which reduces strength of neat cement paste should be expected to reduce its durability.

Though for many years responsible for the development of high-early-strength portland cements and high-grade concrete in Europe, I never claimed that the high strength of concrete per se was a guarantee for its resistance to frost, and I was the first to oppose Professor Graf, when he stated that it was the minimum in compression strength of 150 kilos per cu. cm. (2133.5 psi) that gave such a guarantee. Still, low strength

per se is different from a lower strength due to a considerable decrease caused by any chemical, physical or mechanical action. If that happens either to metals or building materials it is always a warning to be very cautious. Since some well known admixtures to concrete improve durability and strength as well, there is no reason for taking any risks, before having carried out further tests on the influence of the air-cement ratio on the properties of concrete as has been done with regard to the water-cement ratio. If we do so we shall soon reach the state 3 as to my forementioned parallel namely:

3. Concretes designed as to the latest development with well balanced limitations for the water-cement ratio and the air-cement ratio as well, both to be determined by volume in freshly made concrete and also after hardening. It might be advantageous to show the percentages of cement, aggregates, water and air, the ratios between them and some properties of the concrete owing to these ratios in a single diagram such as my 4-material parallelogram for concrete in Fig. 3 and Fig. 4 of my paper on Special Cements submitted to the Congress on Large Dams, Washington 1936. Of course, the permissible amount of air voids in concrete should be checked not only with regard to its resistance to frost but also with regard to decrease in strength and resistance to abrasion and to all other important properties required for high-grade concrete. Only admixtures should be used for which the advantages outweigh very much the disadvantages or which are known to have no disadvantages at all.

We have to be grateful to all those who, like Mr. Kennedy, have contributed so much to the solution of this still very complex problem and I would like to add some more comments to the conclusions of his very interesting paper.

To 1(a). If that means that the failure of an air entrainment agent was due to the fact that it did not improve the consistency of neat cement paste, I would like to add that agents for air entraining only are not expected to improve the consistency of cement paste even from the theoretical point of view whilst certain admixtures used for improving the consistency and workability of concrete improve the consistency and workability of neat cement paste too.

To 1(b). The tests described in the paper are very instructive. I made out some similar tests on mixtures of standard sand and water which contained a much lower percentage of water than according to the paper thus more corresponding to the water content usually wanted for the fine sand in concrete. I found in this way that some air entraining agents such as stearine and oleine did not improve the workability of the sand water mixture, did improve the consistency and workability of concrete.

To 2. I agree that the improvement in durability is not due to the air entrainment agent. In my opinion it is due to the effect of the entrained air on the formation of size, shape and distribution of voids. When the entrained air is removed again by applying a vacuum, the voids still left change their character very unfavorably by the procedure of sucking, as already mentioned.

To 3. I agree that well designed rich concrete mixes of appropriate consistency and workability are not improved by adding agents for air entrainment only but they are improved when special admixtures for improving consistency and workability are added.

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Wire-Wound Prestressed Concrete Pressure Pipe*

By RAY B. CREPPS†
Member American Concrete Institute

SYNOPSIS

This paper describes briefly the manufacture of and a simplified design for a concrete shell reinforced by wrapping a prestressed wire around it to give the pipe structural properties for water lines in which high hydrostatic pressures can be carried. A hint as to possible use of pipe of this type for culvert purposes is included.

Conventional reinforced concrete pipes have been used in water supply lines for many years within limitations of internal pressures and external loadings; but with the comparatively recent developments of the wire-wound prestressed concrete pipe greater internal pressures can be carried as well as a greater resistance to external loadings afforded. Two important factors have contributed to the development of the wire-wound prestressed concrete pipe: (a) advances in the production of dense, low water-cement ratio concrete, and (b) production of very high strength steel wire. The process of making prestressed concrete pipe by the Lewistown Pipe Company utilizes these two factors in producing quality pipes for designed internal pressures.

In the production of the concrete for the structural shell of the pipe, fine and coarse aggregates are proportioned for denseness, and a liberal amount of portland cement is used. Sufficient water is added to give the workable concrete a slump of about 5 in. In the production of a cylindrical shell by the spinning process, the concrete is shoveled into the ends of the horizontal revolving steel forms where it melts away against the sides. When sufficient concrete is placed, the forms are speeded up and kept running at a peripheral speed of about 40 miles per hour until the solids of the concrete have forced the water to the inside. Upon slowing down, the water wastes out of the ends of the form. Fig. 1

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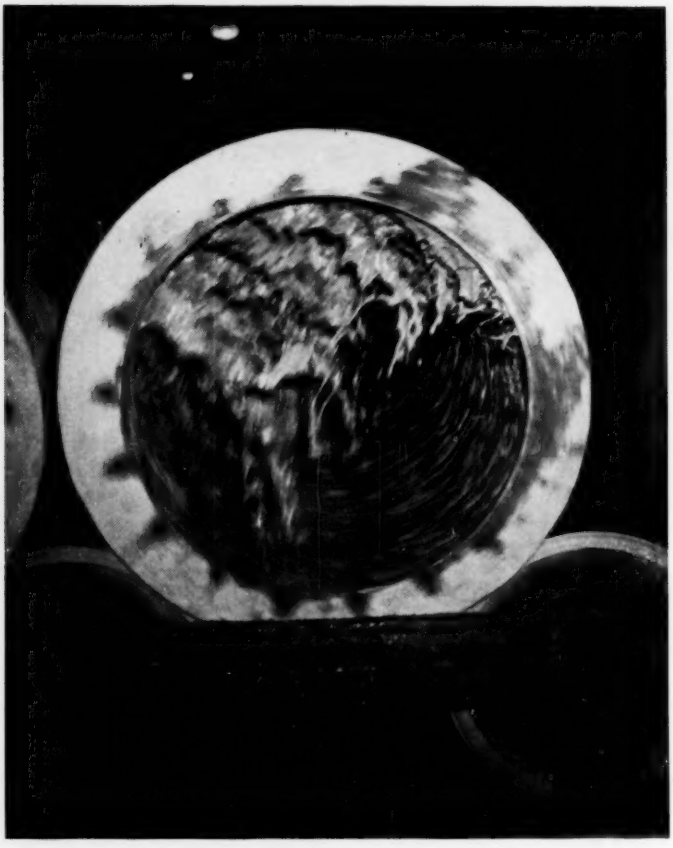

Fig. 1—Excess water being forced out of a 36-in. pipe as the form is being decelerated

shows the excess water forced out of the concrete as the revolving forms for a 36-in. pipe are being decelerated. For shells with a wall thickness greater than 2 in. the concrete is placed in two layers. The inner pipe surface is then smoothed with a trowel-like tool to an even surface.

The steel forms are in two sections for easy removal from the spun concrete cylinder as well as for convenience in the original assembly. End rings of the desired shape are held onto the steel forms by long rods with threaded ends. These longitudinal rods, spaced on about 6-in. centers and extending the entire length of the form, are tightened with a stress-wrench to a designated amount and are incorporated in the concrete when the shell is spun. The end rings support the weight of the 12-ft. length pipe sections during the spinning operation.

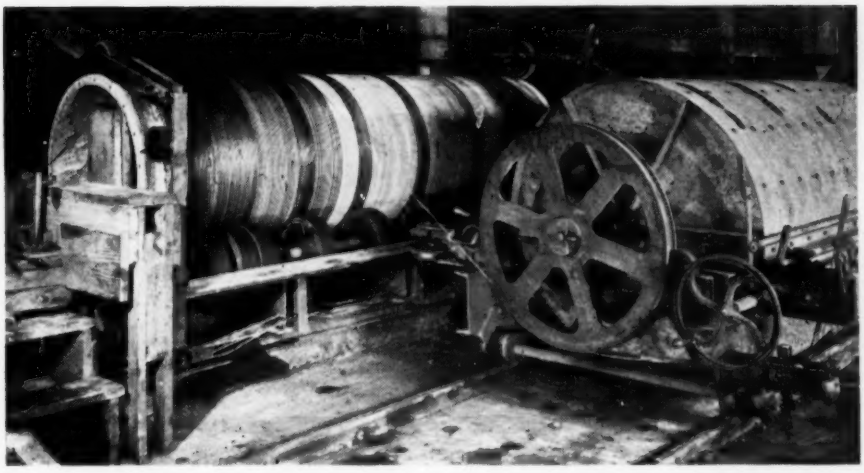

Fig. 2—High strength steel wire under tension is being wrapped around a 36-in. concrete shell by the Lewistown winding machine

From the spinning machine the filled forms are rolled on rails into the steam kiln; the forms are removed the following day. Before the pipe shell is wrapped with steel wire the concrete may attain a compressive strength of 7000 psi or greater, and possesses a high density. Approximate 3-in. cubes sawed from a 36 x 3-in. pipe shell, produced for a Chicago project using crushed stone coarse aggregate, gave compressive strengths of 12,000 psi and a density of 159 lb. per cu. ft.

The object of wrapping the concrete shell on the outside with wire under tension is to produce compressive deformations or stresses in excess of those created in the opposite direction when internal pressure is placed within the pipe. The operation of securing a tension in the steel wire and winding the wire onto the pipe with a designated spacing is accomplished by the Lewistown winding machine. Fig. 2 shows the machine winding a pipe shell—36-in. inside diameter, 3-in. wall thickness, by 12 ft. long.

The shell may be taken directly from the curing kilns or from the storage yard. In either case after the concrete has cooled to room temperature, the shell is placed in the wire winding machine upon a number of rubber covered wheels supporting the pipe along two elements. These wheels and the shafts upon which they are fastened are driven by external power at the end of the machine, causing the pipe shell to be revolved about a horizontal axis.

One end of the high strength steel wire is quickly fastened in a small malleable iron casting previously incorporated near the end of the shell during the spinning operation. As the shell is revolved the wire is wound

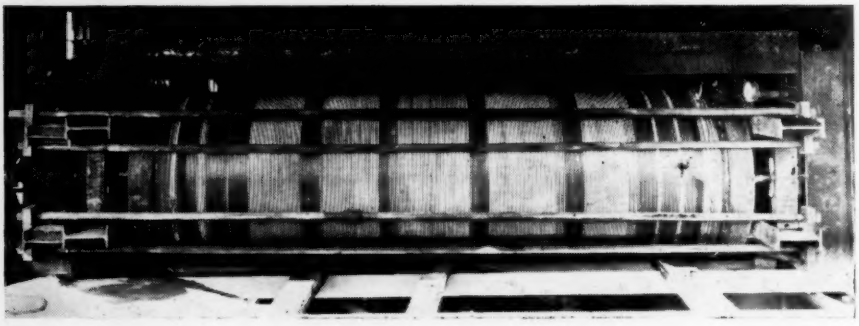

Fig. 3—Assembly for a hydrostatic test of a 36-in. prestressed concrete pressure pipe

spirally along its length and anchored in an iron casting at the far end. The ends of the concrete shell are wrapped heavier than the body. Tension is obtained in the wire by the resistance offered in the large drum of the winding mechanism as the wire is pulled off of the tapered wheel secured to the drum (Fig. 2). A coil of wire is fed onto the tapered wheel and through friction of the several turns of wire the winding mechanism is operated. A number of shelves attached to the interior surface of the drum lift the weight of the dry sand contained therein to give a portion of the wire tension while the force needed to drive the car along the supporting track gives the remainder. Replaceable gears are connected to the sand drum and the car in order to move the car along the track to produce the desired spacing in winding the wire on the concrete shell.

Each prestressed concrete shell is then subjected to internal hydrostatic pressure equal either to the designed working pressure for the pipe, or the local city water pressure, whichever is greater. One pipe, selected at random from each day's production, is subjected to a higher pressure. A general view of the testing assembly is seen in Fig. 3. The rods and I-beams take care of the end thrust from the bulkheads which contain rubber gaskets. For pressures up to 100 psi the rods are not necessary. In Fig. 4 there is seen a 36-in. pipe being subjected to a hydrostatic pressure of more than 200 psi as noted by the position of the pointer in the pressure gauge.

The bell end of each pipe length is formed by mounting a steel band only slightly larger than the outside of the concrete shell, together with a shaped gasket, on one end of the pipe. A cross-section of the gasket appears like a short-toothed comb with inclined teeth and as any hydrostatic pressure comes onto the gasket the inclined fins are pressed tighter against the concrete shell (Fig. 5). The shape at the end of the pipe where the gasket is attached is such that the free end of a pipe 12 ft. long,

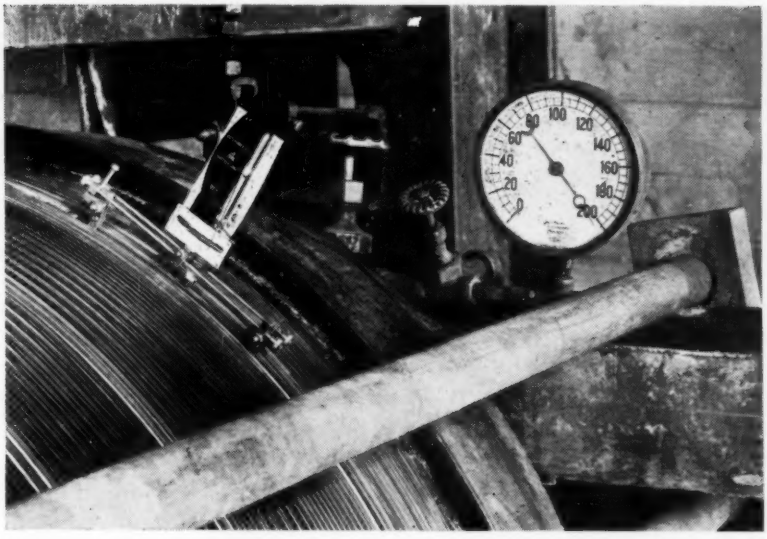

Fig. 4—Internal pressure and increase in steel stress are being measured in a 36-in. pipe

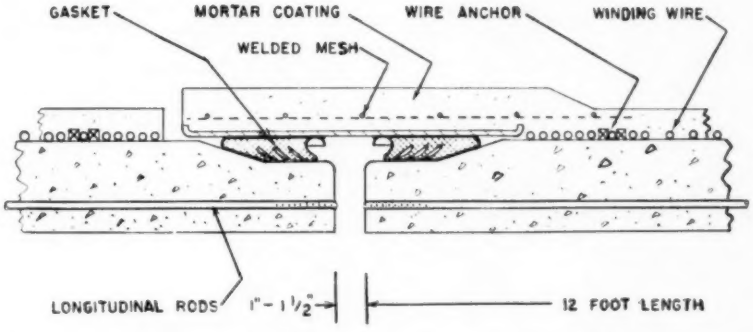

Fig. 5—Schematic arrangement of flexible connection for wire-wound prestressed concrete pipe

being placed in a trench, can be swung about 3 in. from the centerline of the previously placed pipe. For larger angles the steel band is cut and welded before being incorporated in the bell end of the pipe. The pipe with its connection is stood on end; a mesh wire is placed around the steel band overlapping the wire windings; and the steel outer forms are set to place ready to receive a 1-in. coating of portland cement mortar. High-frequency mechanical vibrators attached to the forms cause the mortar to flow around the prestressed steel wire winding. The mortar coating is steam cured.

GENERAL THEORY

The inherent weakness in concrete is its lack of resistance to tensile forces or deformations. Concrete cylindrical shells under three conditions of reinforcement, namely, (a) none, (b) bonded as in culvert pipe, and (c) prestressed wire wound around it, would be able to resist internal hydrostatic pressures as follows:

(a) Only to the extent of the tensile strength, or the corresponding tensile deformation, developed by the concrete,

(b) Only to the extent of the limiting tensile deformation in the concrete and in the steel, limited to the deformation when the concrete would crack, and

(c) To the extent of tensile deformation in the steel wire, the precompressive deformation in the concrete caused by winding the wire under tension onto the shell, and the limiting tensile deformation of the concrete.

Simplified conceptions of the circumferential stress condition in a concrete cylindrical shell with wire wrapping and with subsequent internal pressure are shown in Fig. 6. Employing the usual nomenclature the following formulas are developed.

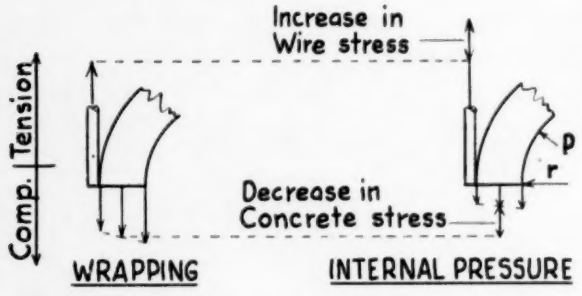

Fig. 6—Simplified conception of the circumferential stress condition in a concrete cylindrical shell with wire wrapping and subsequent internal pressure

f_s = stress in steel, psi
f'_s = stress in steel due to internal pressure, psi
f_c = stress in concrete, psi
f'_c = reduction in concrete prestress, psi
A_s = Area of steel, sq. in.
A_c = Area of concrete, sq. in.
r = Internal radius of pipe, in.
t = Wall or shell thickness, in.
P = Internal pressure, psi
E_s = Modulus of elasticity of steel, psi
E_c = Modulus of elasticity of concrete, psi
$n = E_s/E_c$
δ_s = deformation in steel, in. per in.
δ_c = deformation in concrete, in. per in.

1. When the concrete shell is wound with a wire in a tensioned state there is produced

(a) a tensile stress in the steel wire, f_s, and

(b) a compressive stress in the concrete, f_c.

Although the wall thickness of the shell is relative thin yet the stress towards the inside surface is slightly greater than that developed towards

the outside surface. This characteristic is shown in Fig. 6. In order to simplify the calculations, the average stress across the shell section is balanced against the stress in the steel.

$f_c A_c = f_s A_s$ For 1-in. pipe length, $A_c = t$

$f_c = f_s A_s / t$ ..... (1)

2. When the prestressed pipe is subjected to internal pressure there is produced:

(a) an increase in the steel prestress, and

(b) a decrease in the concrete precompression.

$Pr = f'_s A_s + f'_c A_c$

$\delta'_s = f'_s / E_s$ and $\delta_c = f'_c / E_c$

But $\delta_s = \delta_c$, therefore $f'_s / E_s = f'_c / E_c$

Whence $f'_s = f'_c E_s / E_c$ Since $E_s / E_c = n$,

$f_s = n f_c$, ... (2)

which is the increase in steel stress accompanying the decrease in the concrete precompression due to internal pressure. Since the concrete is assumed to take no tensile stress the limit of f'_c is the magnitude of the concrete prestress, f_c, whence

$Pr = n f_c A_s + f_c A_c$ For 1-in. length, $A_c = t$

$P = f_c (n A_s + t)/r$, or $f_c = \dfrac{Pr}{n A_s + t}$ ..... (3)

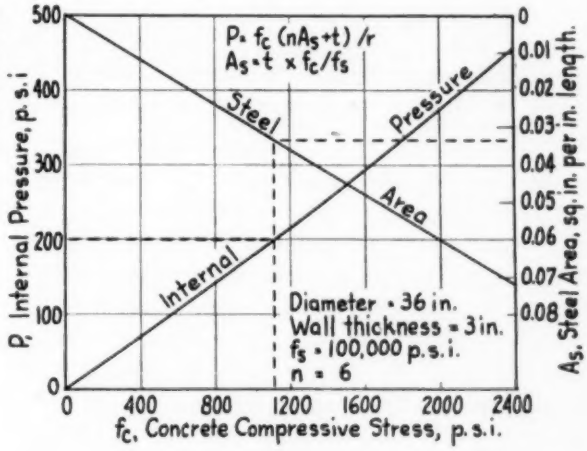

Fig. 7—Typical diagram showing steel area, precompression in concrete shell, and internal pressure to release the concrete precompression to zero stress

A typical diagram based upon the foregoing development for a 36-in. pipe with a 3-in. wall thickness is shown in Fig. 7. The diagram shows the internal pressure required to reduce the induced circumferential precompression to zero stress; and also the steel area required to produce the

concrete compression when the steel prestress is 100,000 psi. Should any other prestress be used, adjustments in steel areas can be made quickly upon an inversely proportional basis. A safety factor is provided in that considerable internal pressure would be required before the high strength concrete shell would develop sufficient tensile deformation for rupture.

To illustrate: For a wrapping steel prestress, f_s, of 100,000 psi and a modulus of elasticity of concrete, E_c, of 5,000,000 psi, an internal pressure of 200 psi, or pressure head of 462 ft., would be required to reduce the circumferential concrete precompression stress of 1120 psi to zero. The steel area is .0335 sq. in. per in. length of pipe, or .402 sq. in. per ft. length, or the equivalent of a No. 6 gauge wire spaced $\frac{7}{8}$ in.

In the design of the wire-wound pipe, advantage is taken of the high steel stresses and for the illustration above, 100,000 psi is shown. This value is very conservative for steels which have a tensile strength of 180,000 psi or above. To the initial steel prestress used in tensioning the wire, there is an additional stress due to the internal pressure, but this increase is very small by comparison. For the above example the increase in steel stress would be: $nf_c = 6 \times 1120 = 6{,}720$ psi, thus making the total steel stress only 106,720 psi, a value well below the proportional limit of the steel. A recent hydrostatic test was made on a 36-in. pipe having a 3-in. wall thickness and a wrapping of steel wire spaced on $\frac{1}{2}$-in. centers. Measurements for the increase in length of the steel wire were taken with a Huggenberger straingage when the internal pressure was raised to 200 psi. See Fig. 4 for an illustration of the method of testing. The strain or increase in length of the wire per in. length was 0.000182 in. which corresponds to an increase in tensile stress in the wire of 5,460 psi. The concrete had a modulus of elasticity of about 6,000,000 psi.

The magnitude of the compressive deformation and the stress created in the concrete are controlled by the spacing of the prestressed wire and are greater for the larger internal pressures to be resisted; thus, from Fig. 7, a compressive stress in the concrete of 1600 psi would be induced in order that the pipe would resist an internal pressure of about 300 psi. During the elapsed time from the wrapping operation until the pipe is placed in service, it may be subjected to drying and to plastic flow. Accompanying the decrease in circumferential deformation from these causes, the wrapping steel prestress may be only slightly lowered. Since the rich concrete in the shell has high density, a low W/C ratio, a high strength, and a high modulus of elasticity it is doubtful if the loss in tension prestress in the wire, over an extended period of time before the pipe would be placed in service would exceed 3,000 to 5,000 psi, or an equivalent reduction in the internal pressure capacity of about 15 psi. However these negative effects will be off-set when the pipe is placed in

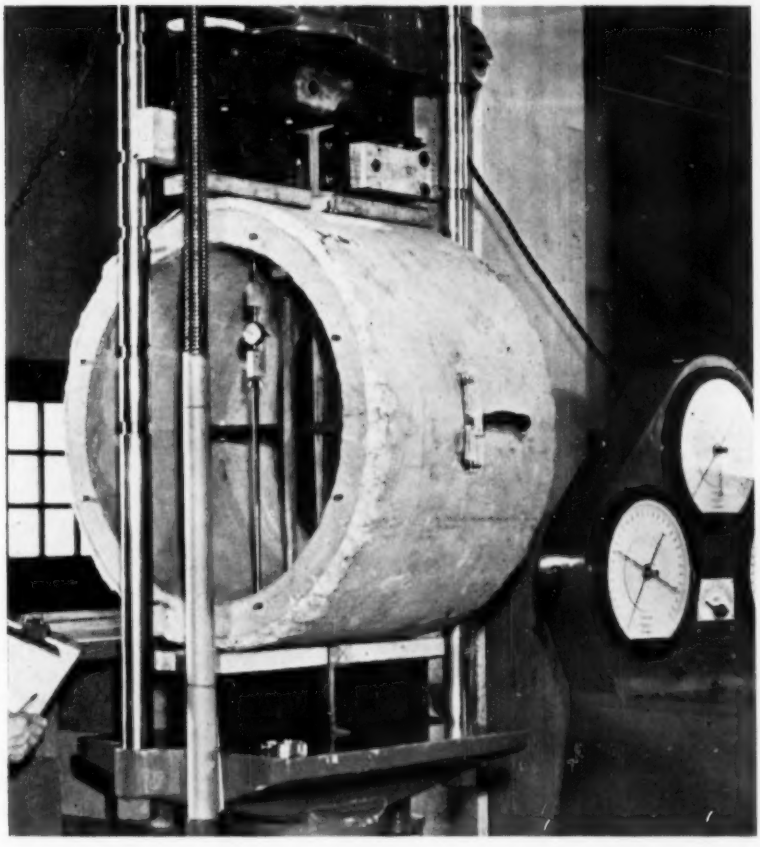

Fig. 8—Two-foot length of 30-in. wire-wound prestressed concrete pipe ready for a three-edge loading test

service for there will result a circumferential expansion due to an absorption of water, a reduction in the concrete compression from the pressure, and an increase in the wire tension.

Accompanying the circumferential compressive deformation created in the concrete shell during the winding process, there is produced a longitudinal deformation. The magnitude of this deformation is small but may amount to a value corresponding to Poisson's ratio of about 0.2. The longitudinal rods which hold the end rings in position during the spinning operation in the Lewsitown pipe are of such size and quantity and are prestressed to a degree so that when the concrete shell is removed from the form a longitudinal deformation is produced to balance that created by the contemplated circumferential wire winding.

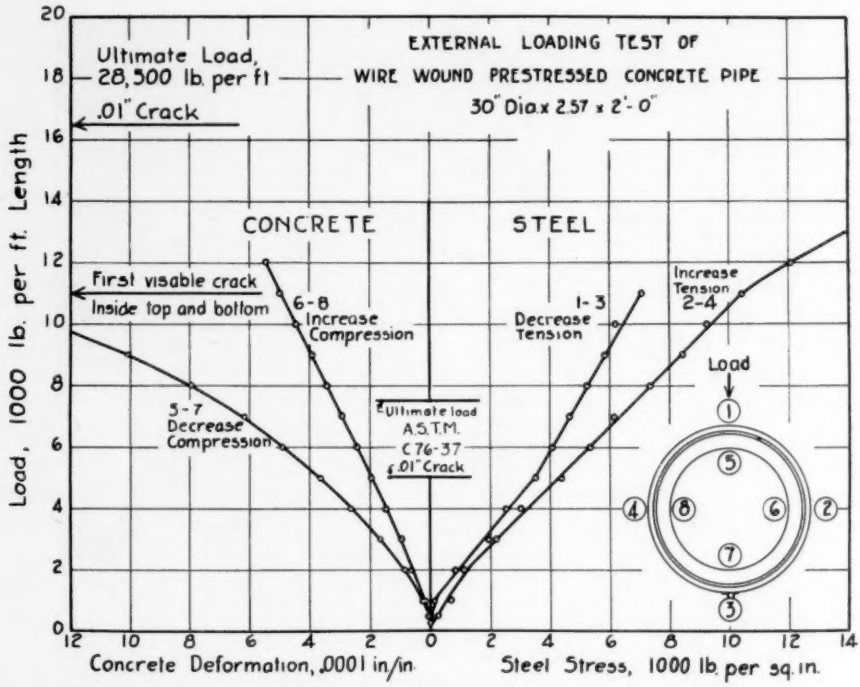

Fig. 9—Diagram showing the results of a three-edge loading test of a 30-in. wire-wound prestressed concrete pressure pipe

The mortar protective coating placed on the pipe is very effective in holding the winding wire in position. In attempts made to remove the wire from an uncoated concrete shell it has been found that a light force from the fingers will be sufficient to hold a wire in position so that the friction of the wire on the shell will develop the full force used in wrapping within one complete turn of the wire around the pipe.

The merits of the wire-wound pipe under external loading are illustrated in a three-edge loading test made in the Materials Testing Laboratory at Purdue University. A 30-in. diameter pipe, 2 ft.-0-in. long with a 2.57-in. structural shell was wrapped with wire under a tension of 83,000 psi. The steel area was 0.039 sq. in. per in. length of pipe. The arrangement for securing deformations in the concrete and steel are noted in Fig. 8 and the plotted results are shown in Fig. 9. The 0.01-in. crack occurred at a load of 16,500 lb. and the ultimate at 28,500 lb. per ft. length of pipe. In comparison with ASTM specifications for Reinforced Concrete Culvert Pipe (C 76-41) the wire-wound pipe had a smaller wall thickness and the quantity of circumferential steel equal to about 1/6 that of elliptical

reinforcement, but it gave loads greater than three times the requirements.

The wire-wound prestressed concrete pressure pipe has advantages of economy of steel and quality of concrete to satisfy engineering designs for high pressure heads. The magnitude of the internal pressure, resulting from hydrostatic heads or external loadings, to be resisted by the pipe will govern the design for quantity of steel wire and its prestress used in wrapping. The designed structure also possesses resistance to external loading exceeding any present requirements for bonded steel reinforced pipe.

Discussion of this paper should be submitted in triplicate to the ACI Secretary before September 1, 1943, for publication in the JOURNAL Supplement concluding Proceedings V. 39.

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Bureau of Reclamation Practice in Design of Joints for Concrete Buildings*

By SAMUEL JUDD†

SYNOPSIS

An illustrated description of the design and construction practice of the Bureau of Reclamation in providing joints in reinforced concrete buildings: (1) between large power units; (2) at large changes in cross-section; (3) at junctions between parts of structures on foundations of different bearing value; (4) at angles between portions of buildings; (5) where buildings are weakened by openings; (6) where concrete placing is interrupted.

In designing large reinforced concrete buildings for power plants and pumping plants, the Bureau of Reclamation recognizes the importance of providing appropriately placed joints in the concrete to facilitate construction and prevent destructive or unsightly cracking. Joints are also used to separate buildings or parts of a building which, in service, would harmfully transmit to each other vibratory stresses or thrusts if not so separated.

Although recent articles in engineering and architectural magazines indicate that progress is being made in joint design, it is believed that further studies and experiments are needed before the questions that so frequently confront the designer are reduced to a practical minimum.

From the standpoint of utility, joints in concrete buildings may be classified under four principal types: "Construction," "contraction," "expansion," and "control." Frequently one joint is a combination of two or more of these types.

Joints in buildings designed by the Bureau are provided: (1) between large power units; (2) when large changes in cross section occur; (3) at junctions between parts built on foundations having different bearing values; (4) at angles between large portions of buildings, such as occur in L-, T-, H-, and U-shaped structures; (5) where buildings are weakened

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by openings, and (6) where it is necessary to interrupt placing of concrete. Selection of the exact position of building joints is governed by both structural and architectural requirements.

The following descriptions apply to the use, design, and construction of joints for Bureau of Reclamation buildings of the types mentioned.

CONSTRUCTION JOINTS

Construction joints are the result of practical limitations that interfere with continuous placing of concrete. They are junctions produced by placing fresh concrete against surfaces of hardened concrete. Construction joints are usually, but not necessarily, vertical or approximately horizontal; in either case, the reinforcement steel is continued across the joint, and care is taken to obtain good bond between the two casts of concrete.

It is often necessary to take special precautions to insure watertightness in construction joints that will be in contact with water, especially those subjected to hydrostatic pressure. Under such conditions, vertical joints need effective sealing because of their tendency to open up as the concrete shrinks. In general, a horizontal construction joint in a wall does not require a seal, if the hardened concrete face has been properly cleaned and roughened so as to insure tight bonding of the subsequently placed concrete.

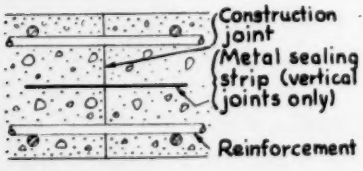


Fig. 1—Construction joint

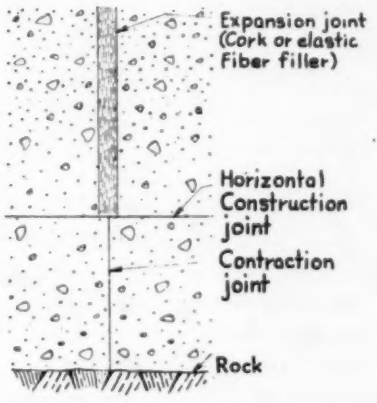
Seals used by the Bureau consist of strips of noncorrodible metal, commonly 9 in. wide and 0.0375 in. thick. They are usually placed in the center of the wall (or slab), parallel to the face of the concrete and at right angles with the joint. They extend equal distances through each side of the joint, and are welded or brazed together at the ends to form a continuous diaphragm. This type of joint is shown in Fig. 1. Careful molding of the concrete around the metal seal is important.

CONTRACTION JOINTS AND EXPANSION JOINTS

The arrangement of main units in hydroelectric power plants and pumping plants necessitates the design of long, narrow buildings which require joints not only to prevent objectionable cracking, but also to allow for expansion and insure against misalignment of machinery that

would otherwise result from building distortion. In the larger plants, such as Boulder, Shasta, and Grand Coulee, joints are placed between main units. In effect, the plant is divided into a series of buildings, each housing a complete power unit. These separations, which are between 57 and 65 feet apart in the plants mentioned, begin at the foundation and extend through the entire buildings, dividing the walls, floors, ceilings, and roof.

Fig. 2—Expansion and contraction joints



Contraction joints are used to relieve tensile stresses induced in a concrete structure by shrinkage. They are commonly used where temperature variations are very small, and volume change in the concrete is confined to shrinkage. They differ from construction joints in that means are used to prevent bond between the joint faces and the reinforcement is discontinuous at the joint. Concrete on one side of the joint is cast first and, after the form is removed from the joint face, a coat of paraffin is applied to prevent adhesion of the concrete placed against it. A metal seal is installed if watertightness is necessary, the seal being provided with a fold to permit opening of the joint without tearing the metal. Obviously, contraction joints (see lower portion of Fig. 2) may also serve as construction joints.

Expansion joints eliminate or greatly reduce compressive stresses in concrete that would otherwise result from thermal expansion and might crush, buckle, or crack parts of the structure. A space, usually one inch, is left between the concrete joint faces to permit expansion and allow room for joint filling and seals. Expansion joints might more appropriately be designated "expansion and contraction" joints as they normally function so as to serve the dual purpose. They also serve as construction joints, and are sometimes used below grade in lieu of simple contraction joints.

Expansion joint filler is usually a waterproof, elastic, pre-molded material such as corkboard or fiberboard. After the forms have been removed, the concrete joint face cast first is coated with an asphalt or bituminous emulsion high in plasticity and adhesiveness. The joint filler is pressed in place against this coating, and then coated on the other side, after which concrete placing is resumed.

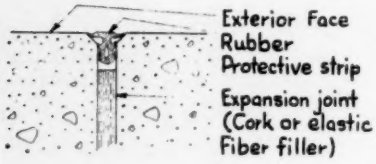


Fig. 3—Rubber protective strip—wall type

Covers are commonly inserted to protect exposed edges of expansion joints, to prevent ingress of foreign material that might otherwise cause spalling of the concrete edges, and to improve the appearance of the structure. At Boulder Dam Power Plant, wall and ceiling joints are covered with aluminum strips, and the floor joints have protective strips of aluminum on each side and rubber tubing wedged between them. A rubber protective strip has recently been developed and used for joints in floors, walls, and ceilings. This cover (Fig. 3) is easily pressed into place and compensates for movement either at right angles with or parallel to the joint. It may be manufactured in color to match the concrete.

Where necessary, expansion joints are made watertight by use of non-corrodible metal sealing strips and asphalt seals. The metal strips provide for movement in horizontal directions, as indicated in Fig. 4. Two strips are installed, one on each side of a diamond-shaped hole which is filled with asphalt or bituminous material as an additional seal against possible leaks in the metal strips. The filler is heated when installed, to insure a dense filling, thoroughly molded into all angles and crevices. Heating is accomplished by steam pipes, which are left in place and may be used for reheating if leaks should develop later. As an additional precaution, drains are constructed behind the metal seals.

Choice of appropriate metal for a joint seal is important. The seal must be flexible, nonbrittle, easy to weld, and noncorrodible. Its cost must also be considered. At the Boulder Dam Power Plant, copper-nickel alloy (monel metal) was used, and at Grand Coulee the seals were of stainless steel. Steel alloy should be of low carbon content, or stabilized by additions of columbium or titanium, otherwise welding will destroy resistance to corrosion in the area of the weld.

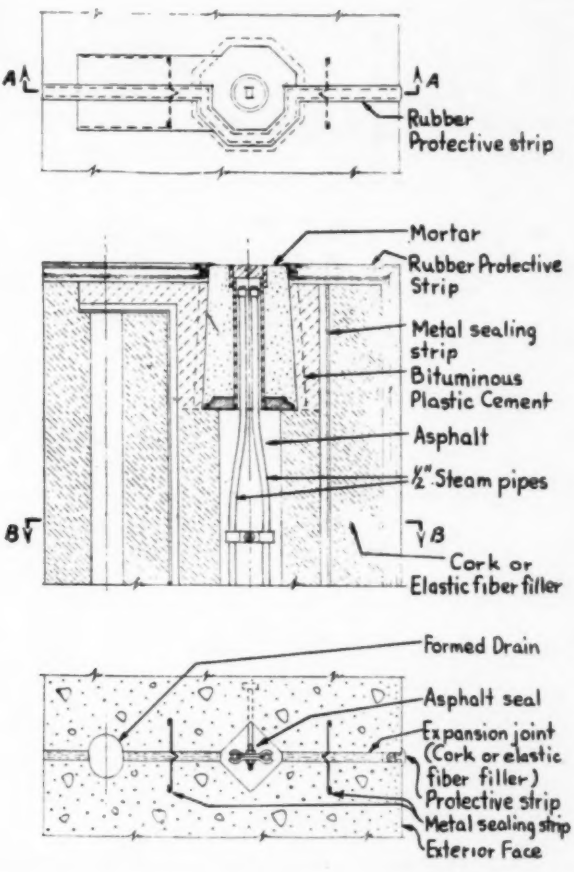


Fig. 4—Expansion joint seals: top—plan at top of joint; center—section A-A —see plan; bottom—Section B-B

As indicated in Fig. 5, the metal seal is tacked to the form in such a manner as to embed one side of the strip in the side of the joint first placed. After the form is removed, the seal is straightened to its final shape for embedment in the concrete on the otherside of the joint.

CONTROL JOINTS

Prior to the adoption of control joints, the Bureau experienced considerable difficulty in keeping concrete walls of unjointed buildings free from unsightly cracking in areas adjacent to openings. Diagonal reinforcement bars at corners of openings proved to be only partially effective.

Satisfactory solution of the problem was found in the use of control joints (Fig. 6), which are planes of weakness along which cracking may take place without marring the appearance of the building. The joints are usually placed at regular intervals along the building walls and at the center of openings or groups of openings, at both top and bottom. The

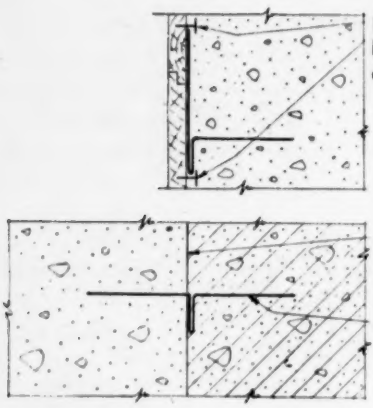
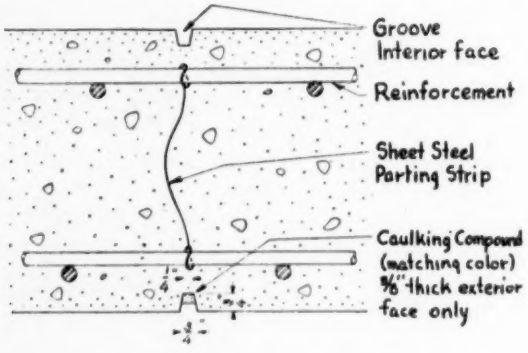


Fig. 5—Metal seals—first pour, metal seal held against form board by nails

Fig. 6—Control joints



parting strip shown in the figure is 20-gage, black sheet steel, waxed on one side, and wired to the reinforcement bars or to the form. Narrow grooves are formed at the wall faces in line with the parting strip to further weaken the section and insure an inconspicuous position for the crack. Exposed grooves are filled with plastic cement, colored to match the concrete.

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prepared under the supervision of J. L. Savage, Chief Designing Engineer, and W. H. Nalder, Assistant Chief Designing Engineer. All engineering and construction work is under the general direction of S. O. Harper, Chief Engineer, and W. R. Young, Assistant Chief Engineer. All activities of the Bureau are under the direction of John C. Page, Commissioner, Washington, D. C.

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The Plasticity Ratio of Concrete and Its Effect on the Ultimate Strength of Beams*

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Member American Concrete Institute

SYNOPSIS

The hypothesis is advanced that the stress-strain diagram for concrete under short-time loading may be idealized for certain purposes so as to consist of two linear parts, one representing elastic behavior and the other representing plastic behavior. The former is measured by the "modular ratio" which is defined as the ratio of the modulus of elasticity of steel to the initial modulus of elasticity of concrete. The latter is measured by the "plasticity ratio" which is defined as the ratio of the plastic strain to the total strain at rupture of the concrete. An empirical equation is given to express the relationship between the plasticity ratio and the compressive strength of concrete made with gravel or crushed stone aggregate. Formulas are derived for the ultimate strength of beams reinforced in tension only. From published results of tests of beams which have failed by compression, experimental evidence is presented which justifies the concept of the plasticity ratio as a function of the compressive strength of concrete.

1. INTRODUCTION

In recent years the criticisms of the ordinary straight-line theory of reinforced concrete have given rise to proposals for the abandonment of the modular ratio, $n = E_s/E_c$, in favor of formulas based on various theories of plastic action.[1], [2], [3], [4], [5]‡ In all of these theories emphasis is placed upon the load at final rupture of the beam. While the wisdom of such emphasis is open to question, there can be no doubt that it is desirable to be able to predict, within reasonable limits, the ultimate capacity of a beam.

In this paper there is described a concept of the action of concrete in compression which permits the derivation of formulas for the ultimate resistance of beams. The concept involves a simplification of the actual

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‡See references at end of paper.

behavior of concrete into an idealized behavior which may be described graphically by a stress-strain diagram consisting of two straight lines, one representing elastic action and the other representing plastic action. A measure of the slope of the line representing elastic action is given by the initial modulus of elasticity of the concrete or by the ratio, $n = E_s/E_c$. A measure of the extent of the plastic action is given by the ratio of the plastic to the elastic strain and is called the "plasticity ratio."

In terms of the usual properties relating to the strength of concrete, f'_c and E_c, and in terms of the plasticity ratio, formulas are derived which predict the ultimate strength of rectangular beams reinforced in tension only and loaded so as to undergo constant maximum moment over a portion of the beams. The manner of failure is predicted for any percentage of reinforcement provided that there is adequate provision against failure by bond or diagonal tension.

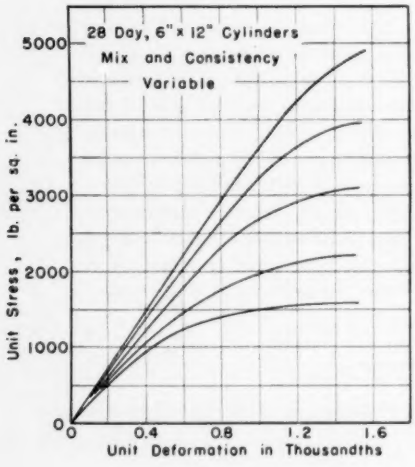


Fig. 1 (left)—Typical stress-strain curves for concrete made with gravel aggregate

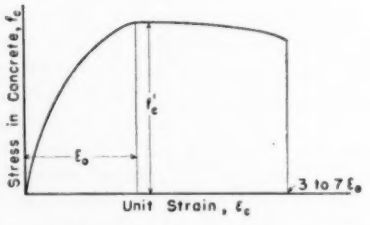


Fig. 2 (above)—Possible stress-strain diagram for concrete at strains approaching rupture—Saliger

2. STRESSES AND STRAINS IN CONCRETE: THE PLASTICITY RATIO

Stress-strain diagrams for concrete of varying strength are shown in Fig. 1. These diagrams are of the usual kind obtained from tests of 6 by 12-in. cylinders moist cured for 28 days and tested in a moist condition. Gravel aggregates were used in making the concrete and the mix and consistency varied for the different strengths. The diagrams illustrate an essential difference between the high-strength and low-strength concretes. For the former, the stress-strain diagram is more nearly linear and for the latter it is more nearly like a second degree parabola up to the maximum stress developed. The point is emphasized that no single shape of curve is typical for all concrete strengths.

Strains beyond those corresponding to maximum stress in the concrete have, until recently, been regarded as of little consequence and have been ignored. Some attention, however, has recently been given to the stress-strain relationship for concrete at strains up to final rupture of the test specimen.[1], [2] The studies which have been made seem to indicate that strains of varying but appreciable magnitude can be developed in the test specimen during a period when the load may be maintained very nearly at maximum value or may be declining at a steady rate with increasing strain. The exact character of this portion of the stress-strain curve for concrete of any strength is not well established. In fact, test data are very meager and are likely to be greatly influenced by the speed of testing.

For strains beyond those which accompany maximum stress, the difference in the curves for various strength concretes is uncertain. One characteristic difference, apparent from tests, is that concretes of low strength fail more gradually than those of higher strength, the strains at final rupture being several times as great for concretes of exceedingly low strength as for those of high strength. The higher strength concretes are likely to fail suddenly, final rupture occurring almost immediately after the maximum load is reached.

For the purpose of explaining the behavior of concrete in a flexural member, a typical set of stress-strain curves extended to rupture for gravel concrete is assumed as shown in Fig. 3(a). These curves are then further idealized in that they are represented approximately by a family of trapezoids as shown in Fig. 3(b). This family of curves is sufficiently broad in scope to cover all grades of concrete from the highest to the lowest strength. The proportions of the trapezoidal diagram for a given compressive strength of concrete, f'_c, depend on two coefficients, namely E_c, the initial modulus of elasticity of the concrete, and β, the ratio of the plastic deformation to the total deformation of the concrete at rupture, defined herein as the "plasticity ratio."

Although a wide scatter is shown by the test data relating to the initial modulus of elasticity of concrete made with gravel or crushed stone, a satisfactory approximation may be made from the following empirical formula for the modular ratio:

$$n = \frac{E_s}{E_c} = 5 + \frac{10{,}000}{f'_c} \qquad (1)$$

where f'_c is the numerical value of the compressive strength, expressed in psi for 6 by 12-in. cylinders. The formula is simple and easy to use. The modulus of elasticity of steel, E_s, may be taken as 30 million psi.

The existence of a property of concrete, measured quantitatively by the plasticity ratio, may be recognized in the behavior of concrete com-

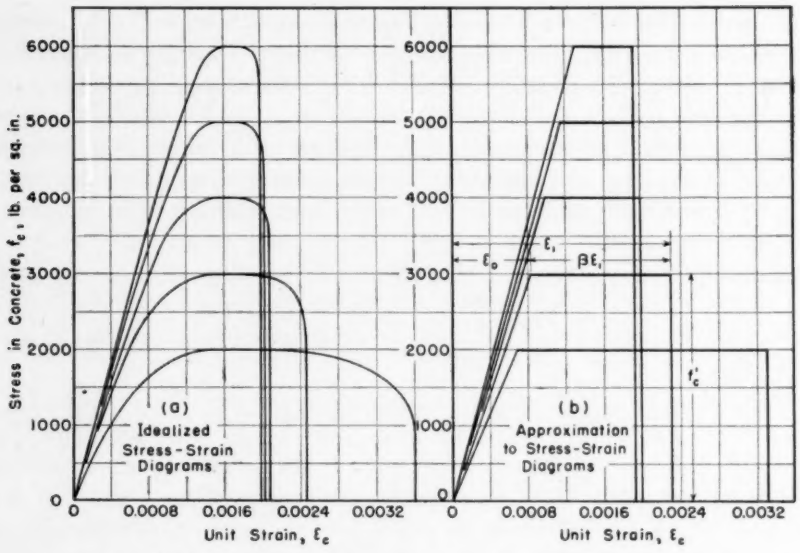


Fig. 3—Idealized stress-strain curves for concrete of different strengths

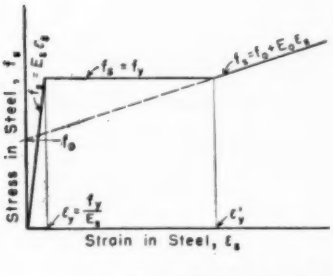
pression specimens at loads near rupture. The sudden rupture of the specimen of high strength, almost immediately after the maximum load is reached, gives evidence that the horizontal portion of the idealized stress-strain diagram is relatively small for concretes of high strength. The gradual breakdown of the specimen of low strength, accompanied by large strains and the maintenance of a considerable proportion of its maximum resistance, gives evidence that the horizontal portion of the trapezoidal diagram is relatively large for concretes of low strength. If ϵ_1 denotes the total concrete strain represented in the trapezoidal diagram at final rupture and $\beta\epsilon_1$ denotes the plastic strain represented by the horizontal portion of the diagram, then β is the plasticity ratio which may have a numerical value within the extreme range between zero and one.

For the purpose of calculating the ultimate resistance of beams, it is proposed that β be determined from an empirical equation of the form

$$\beta = \frac{1}{1 + \left(\frac{f'_c}{4000}\right)^2} \qquad (2)$$

where, as in Eq. (1), f'_c represents the numerical value of the compressive strength of concrete when expressed in psi. The adequacy of this equation will be tested later.

The essential feature brought out here is that, for a given strength of concrete, there are two characteristics necessary to describe the stress-



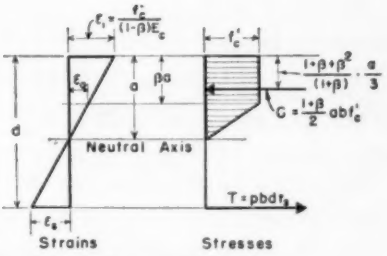


Fig. 4 (left)—Idealized stress-strain diagram for steel
Fig. 5 (right)—Strains and stresses on cross-section of beam

strain relationship all the way to rupture. These are the modular ratio, defined empirically by Eq. (1), and the plasticity ratio, defined empirically by Eq. (2). The complete trapezoidal stress-strain diagram is thus determined by f'_c since n, E_c and β are then known from Eq. (1) and (2), and since the strains defined in Fig. 3(b) are then obtainable from the equations

$$\epsilon_o = \frac{f'_c}{E_c}, \qquad \epsilon_1 = \frac{f'_c}{(1-\beta) E_c} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

3. STRESSES AND STRAINS IN REINFORCING STEEL

For reinforcing steels which possess a marked yield point a portion of the stress-strain diagram may be idealized as shown in Fig. 4. There seems to be no simple law to express the relative extent of the horizontal part of the curve nor to express the slope, E_o, of the diagram to the right of the horizontal portion of the curve. This slope has been variously measured at from one-half to two million psi. The characteristics of the curve near the yield point and beyond are doubtless materially affected by the relative cooling of the bar during rolling as well as by the composition of the steel, the size of the bar, and the speed of testing.

The three linear relations between stress and strain described in Fig. 4 may be expressed as follows:

$$\left.\begin{array}{lll} f_s = E_s \epsilon_s & \text{for} & \epsilon_s < \epsilon_y \\ f_s = f_y & \text{for} & \epsilon_y < \epsilon_s < \epsilon'_y \\ f_s = f_o + E_o \epsilon_s & \text{for} & \epsilon_s > \epsilon'_y \end{array}\right\} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

A complete representation of the diagram shown in Fig. 4 is possible when f_y, ϵ'_y and E_o are measured since it is sufficiently accurate to assume $E_s = 30$ million psi and since

$$\epsilon_y = \frac{f_y}{E_s} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

and

$$f_o = f_y - E_o\epsilon'_y \quad \ldots\ldots (6)$$

Measured values of ϵ'_y between 0.005 and 0.020 are common.

In the discussion of the ultimate strength of beams it will be shown that the steel stress at rupture of the beam may be in any one of the three ranges represented in Eq. (4); that is, at rupture of the beam the stress in the steel may be (1) less than the yield point, (2) at the yield point, (3) greater than the yield point. Saliger[1] pointed out these three possibilities, but gave an unsatisfactory explanation of the theoretical basis for the third condition. The particular condition which will apply to a given beam will depend on the properties of the steel and concrete and on the percentage of reinforcement.

For reinforcing steels which do not have a marked yield point, no sharp classification can be made as to the nature of the steel stress at rupture of the beam. For a given concrete reinforced with such steel the stress in the steel at failure of the beam decreases gradually with an increase in the percentage of steel used as will be discussed later.

4. CALCULATION OF ULTIMATE STRENGTH OF RECTANGULAR BEAMS REINFORCED IN TENSION ONLY

A beam, tested in flexure under gradually applied loads, may fail in a number of ways including diagonal tension, bond, tension in the steel, compression in the concrete, local failure at supports or hooks, and various combinations of these. In this discussion the only types of failure to be considered are tension in the steel and compression in the concrete. In all that follows it is assumed that the beam has adequate strength against diagonal tension or bond failure.

The distribution of strain on a section normal to the axis of the beam is assumed to be linear. By "strain" is meant deformation per unit of length in the transformed section which excludes concrete in tension. The distribution of strain in the concrete from top to bottom of the beam after cracking is, of course, not linear. This non-linearity is, however, of no significance in the analysis. The ultimate load on a beam is assumed to be reached when the compressive strain in the extreme fiber at a cracked section reaches the value ϵ_1 given by Eq. (3), namely $\epsilon_1 = \dfrac{f'_c}{(1 - \beta)\, E_c}$, regardless of the tensile strain in the steel. If the steel yields before the ultimate load is reached, it will continue to deform with increase in load until finally the concrete crushes. Under these assumptions the distribution of strains in the cross section of the beam at rupture may be represented as shown in the left-hand portion of Fig. 5.

Corresponding to the linear distribution of strains in the concrete there is a trapezoidal distribution of stress shown in Fig. 5. If the depth to the neutral axis is denoted by a and the width of the beam is b, it is easily determined that the total compressive force in the concrete is $C = \frac{1+\beta}{2} ab f'_c$ and that its line of action is at the depth given in Fig. 5. If f_s represents the stress in the steel at rupture of the beam, the equality of the total tension in the steel and the total compressive force in the concrete requires that $\frac{1+\beta}{2} ab f'_c = pbd f_s$. From this equation the relative depth to the neutral axis is found to be

$$\frac{a}{d} = \frac{2}{1+\beta} \frac{pf_s}{f'_c} \quad \ldots\ldots (7)$$

The ultimate resisting moment produced by the couple shown in Fig. 5, found by evaluating $M_{ult.} = C\left(d - \frac{1+\beta+\beta^2}{1+\beta} \frac{a}{3}\right)$, is given by the equation

$$\frac{M_{ult.}}{bd^2 f'_c} = \frac{a}{d}\left(\frac{1+\beta}{2} - \frac{1+\beta+\beta^2}{6} \frac{a}{d}\right) \quad \ldots\ldots (8)$$

When (7) is substituted into (8), the resulting equation is

$$\frac{M_{ult.}}{bd^2 f'_c} = \frac{pf_s}{f'_c}\left(1 - \frac{1}{N} \frac{pf_s}{f'_c}\right) \quad \ldots\ldots (9)$$

wherein N is a constant for any strength of concrete and is given by the formula

$$N = \frac{3(1+\beta)^2}{2(1+\beta+\beta^2)} \quad \ldots\ldots (10)$$

For various strengths of concrete, provided that β is related to f'_c according to Eq. (2), the factor N takes values as follows:

f'_c, psi	:	2000	3000	4000	5000	6000
N	:	1.99	1.97	1.93	1.88	1.83

Since pf_s/f'_c is small, it is sufficiently accurate for practical purposes to take $N = 2$. Then Eq. (9) becomes

$$\frac{M_{ult.}}{bd^2 f'_c} = \frac{pf_s}{f'_c}\left(1 - \frac{1}{2} \frac{pf_s}{f'_c}\right) \quad \ldots\ldots (11)$$

Thus far the equations have been perfectly general, applying to all concrete strengths and to all percentages and qualities of steel. However, the numerical value of f_s to be used in Eq. (11) is still undetermined. From the strain relations given in Fig. 5 note that

$$\epsilon_s = \left(\frac{d}{a} - 1\right)\epsilon_1 = \left(\frac{d}{a} - 1\right)\frac{f'_c}{(1 - \beta)\,E_c} \quad \ldots\ldots (12)$$

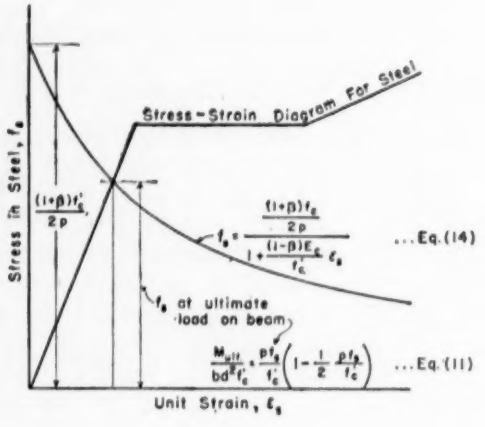
from which

$$\frac{a}{d} = \frac{1}{1 + \dfrac{(1 - \beta)\,E_c}{f'_c}\epsilon_s} \quad \ldots\ldots (13)$$

Equating (13) and (7) and solving for f_s gives

$$f_s = \frac{\dfrac{(1 + \beta)\,f'_c}{2p}}{1 + \dfrac{(1 - \beta)\,E_c}{f'_c}\epsilon_s} \quad \ldots\ldots (14)$$

Fig. 6—Steel stress at ultimate load on beam from Eq. (14) and from stress-strain diagram for steel



This is one relation between f_s and ϵ_s at rupture of the beam for any given strength of concrete and percentage of steel. Fig. 6 shows a typical curve obtained from Eq. (14) for a given value of f'_c and of p. Superimposed on this curve is an assumed stress-strain diagram for steel. Since the relation between f_s and ϵ_s given by each of these curves must be satisfied, the intersection between the two curves gives the value of f_s and of ϵ_s at rupture of the beam.

A graphical representation of the type given in Fig. 6 is helpful in visualizing the behavior of simple beams. Fig. 7 shows families of curves determined from Eq. (14) for a concrete strength of 3000 psi and for percentages of steel varying from 0.2 to 6 per cent. On this figure there are superimposed a number of possible stress-strain diagrams for steel. From the intersections between the family of curves and a typical stress-strain diagram for steel, several inferences may be drawn: (1) For low

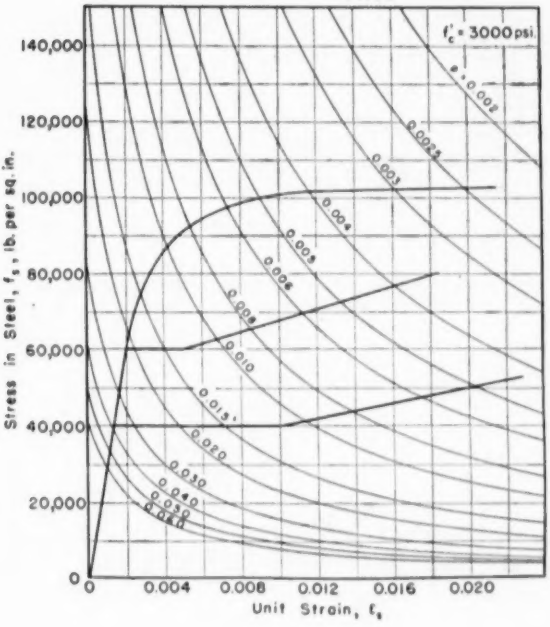


Fig. 7—Relation between stress and strain in steel at rupture of beam by Eq. (14) for 3000-lb. concrete

percentages of steel, the stress in the steel at rupture of the beam may greatly exceed the yield point; (2) Where a given steel has a definite yield point, there is a range of percentages of steel for which the stress at rupture of the beam will be the yield point, f_y; (3) For high percentages of steel, rupture of the beam will occur before the yield point of the steel is reached. Families of curves similar to those shown in Fig. 7 may be drawn for other strength concretes.

To translate the statements just made into quantitative analytical conditions, reference is again made to the idealized stress-strain diagram shown in Fig. 4 for a steel with a definite yield point. When the yield point is first reached the strain in the steel is $\epsilon_s = \epsilon_y = f_y/E_s$ and the stress is $f_s = f_y$. If these relations are substituted into Eq. (14) and if the resulting percentage of steel is donated by p_o, it will be found that

$$p_o = K\frac{f'_c}{f_y} \quad \ldots\ldots (15)$$

wherein

$$K = \frac{(1+\beta)/2}{1+\dfrac{(1-\beta)f_y}{nf'_c}} \quad \ldots\ldots (16)$$

Equation (15), with K given by (16), is interpreted as defining the critical

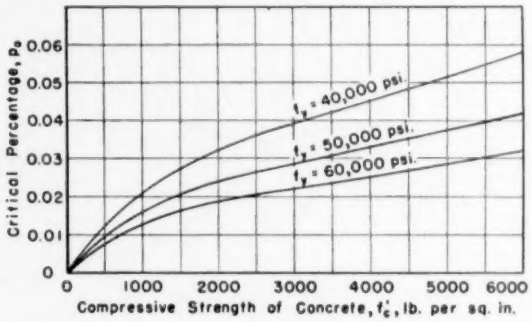


Fig. 8—Critical percentage of steel, p_o, for compression failure

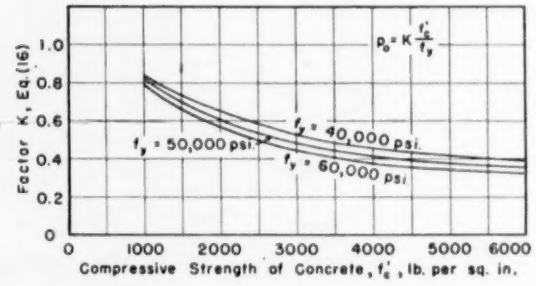


Fig. 9—Factor K to express critical percentage of steel, p_o, as $K\frac{f'_c}{f_y}$

percentage of steel p_o at which a compression failure occurs in the concrete simultaneously with the stressing of the steel to the yield point f_y.

The variation of p_o with f'_c is shown in Fig. 8 for three values of yield point, f_y. The variation of the coefficient K with f'_c and f_y is shown in Fig. 9. It will be seen that the calculated values of K, for a considerable range of common values of f'_c and f_y, agree with the particular values of 0.456 found by Whitney[4] and 0.47 reported by Cox,[5] but that values as high as 0.65 and as low as 0.33 may apply in particular instances.

The steel stress at rupture of the beam will be less than the yield point when the percentage of steel p is greater than the critical percentage p_o. That is $f_s < f_y$ when $p > p_o$ where p_o is defined by Eq. (15). Under this condition the steel is elastic and $f_s = E_s\epsilon_s$. Expressing ϵ_s in this equation by means of (12) and substituting the resulting expression for f_s into (7) gives a second degree equation in a/d which has a solution

$$\frac{a}{d} = \frac{2}{1 + \sqrt{1 + \frac{2(1 - \beta^2)}{np}}} \qquad (17)$$

This is the relative depth to the neutral axis when p is greater than p_o and the failure is by compression. Eq. (17) has been put in the most

convenient form for calculations although it may be written in the form

$$\frac{a}{d} = \sqrt{\left(\frac{np}{1-\beta^2}\right)^2 + \frac{2np}{1-\beta^2}} - \frac{np}{1-\beta^2} \quad \text{(17a)}$$

When the plasticity ratio β is zero this reduces to the well-known expression for k, the relative depth to the neutral axis as given by the ordinary theory.

From Eq. (7) the stress in the steel is

$$f_s = \frac{(1+\beta)f'_c}{2p}\frac{a}{d} \quad \text{(18)}$$

From (17) and (18)

$$\frac{pf_s}{f'_c} = \frac{1+\beta}{1+\sqrt{1+\dfrac{2(1-\beta^2)}{np}}} \quad \text{(19)}$$

When the percentage of steel is greater than p_o, the ultimate resisting moment may be calculated from Eq. (8) using a/d from (17) or may be calculated from Eq. (11) using pf_s/f'_c from (19).

When p is greater than p_o, the variation of $M_{ult.}/(bd^2f'_c)$ with p and f'_c is represented graphically by the portion of the curves shown in Fig. 10 for compression failures. When the initial failure is by compression, rupture may occur suddenly, without warning, if the concrete is of high strength. This is an important fact to consider in relation to the advisability of using percentages of steel as high as p_o.

Consider again the idealized stress-strain diagram for steel as represented in Fig. 4. The steel strain is ϵ'_y at the place where stress just begins to increase above the yield point. If the simultaneous values $p = p_1$, $f_s = f_y$, $\epsilon_s = \epsilon'_y$ be substituted into Eq. (14), the second critical percentage is found to be

$$p_1 = \frac{f'_c}{2f_y}\frac{1+\beta}{1+(1-\beta)\dfrac{E_c\,\epsilon'_y}{f'_c}} \quad \text{(20)}$$

For percentages of steel greater than p_1 and less than p_o yielding of the steel will precede final rupture of the beam. Furthermore, the stress in the steel at final rupture will be the yield point, and the strain in the steel will have a value between the limits ϵ_y and ϵ'_y. Thus, from Eq. (7) and (11), when $p_1 < p < p_o$, the equations which apply are $f_s = f_y$,

$$\frac{a}{d} = \frac{2}{1+\beta}\frac{pf_y}{f'_c} \quad \text{(21)}$$

$$\frac{M_{ult.}}{bd^2f'_c} = \frac{pf_y}{f'_c}\left(1 - \frac{1}{2}\frac{pf_y}{f'_c}\right) \quad \text{(22)}$$

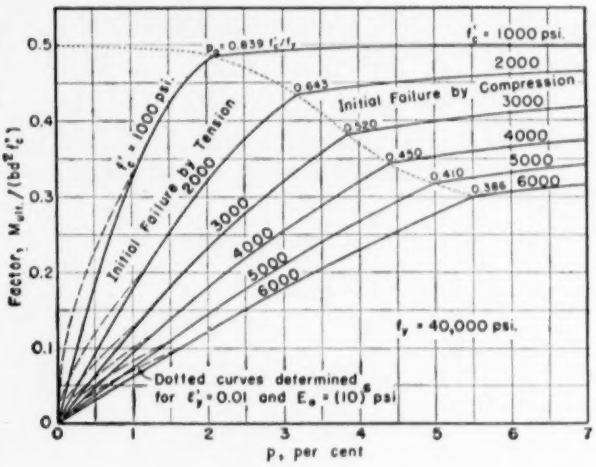


Fig. 10 — Variation of $\frac{M_{ult}}{bd^2f'_c}$ with percentage of steel when yield point is 40,000 psi.

Equation (22) is Saliger's equation. Values of $M_{ult.}/(bd^2f'_c)$ according to Eq. (22) were used to determine the curves shown in Fig. 10 for tension failures in the intermediate ranges of steel percentage when the yield point of the steel is 40,000 psi. For each concrete strength, the critical percentage of steel $p_o = K f'_c/f_y$ is indicated on the figure.

The tensile stress in the steel will be greater than the yield point when the percentage of steel is less than p_1 where p_1 is defined by Eq. (20). Under this condition the stress in the steel is in the range approximated by the equation

$$f_s = f_o + E_o\epsilon_s \quad \ldots\ldots (23)$$

as represented on Fig. 4. By equating (7) and (13) and by substituting ϵ_s from (23) into the resulting equality, a second degree equation in f_s is obtained. Solution for f_s yields the equation

$$f_s = \frac{f_o}{2}\left(A + \sqrt{A^2 + B}\right) \quad \ldots\ldots (24)$$

wherein

$$A = 1 - \frac{1}{1-\beta}\frac{E_o f'_c}{E_c f_o}; \quad B = \frac{2}{p}\frac{1+\beta}{1-\beta}\frac{E_o}{E_c}\left(\frac{f'_c}{f_o}\right)^2 \quad \ldots\ldots (25)$$

The relative depth to the neutral axis may now be determined from Eq. (7) and the ultimate resisting moment from Eq. (11).

For beams which fail at steel stresses above the yield point it must be remembered that the additional carrying capacity above that at first yielding is obtained only at the expense of large deformations and consequently at the expense of large deflections. In Fig. 10 the dotted curves for the low percentages of steel are based on Eq. (11) with f_s given by

(24). Values of ϵ'_y and E_o were assumed as indicated on the figure. Different families of such curves are obtained by making other assumptions regarding ϵ'_y and E_o.

5. COMPARISON OF CALCULATED AND EXPERIMENTALLY DETERMINED ULTIMATE LOADS

Many laboratory tests have been made on beams which have failed in tension or in compression. Most of these have been described quite fully except for some of the detailed properties of the steel which have been noted herein as significant. Lack of knowledge of the extent of yielding of the steel and the rate at which load again picks up is especially unfortunate where low percentages of steel have been used. Tests have shown that some reinforcing bars, particularly of small size such as commonly employed in laboratory tests of beams, exhibit a yield point but then pick up stress almost immediately at a tangent modulus E_o in the neighborhood of 1.0 or 1.5 million psi. Such action is likely to be important in beams having less than 2 per cent of reinforcement. In other bars the amount of yielding at constant stress is very large, the plastic strain being 10 or 20 times as great as the elastic strain.

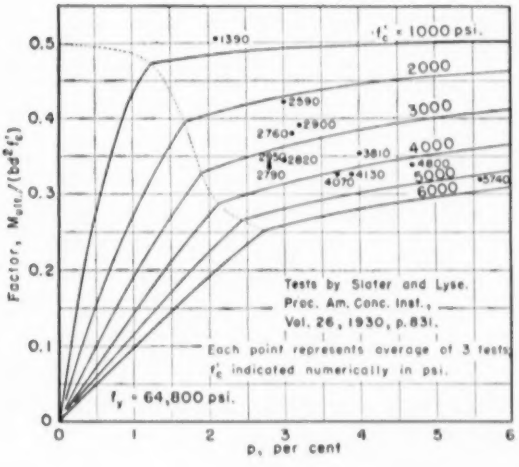


Fig. 11—Comparison of theoretical resisting moments with experimental data

Data pertaining to published tests have been tabulated and resisting moments, calculated according to the equations given herein, have been compared with resisting moments determined by test. While the complete data* are omitted here for lack of space, a few graphical comparisons are made.

In Fig. 11 the plotted points show the results obtained by Slater and Lyse[6] in 1930. The numerical value indicated for each plotted point

*Tabular data may be obtained by interested persons through application to the Secretary of the Institute.

may be compared with the theoretical position given by interpolation between the solid lines. For these tests the ratio of ultimate resisting moments, $M_{test}/M_{calc.}$, varied from 0.90 to 1.10 with an average of 1.00 for a range of concrete strengths from 1390 to 5740 psi and a range of steel percentages from 2.1 to 5.6 per cent. According to the calculations, all of the beams were reinforced in excess of the critical percentage p_o, and, according to the tests, all of the beams failed by compression. The diagram shows how a fortuitous choice of concrete strengths and steel percentages will result in a constant value of $M_{ult.}/(bd^2f'_c)$ for compression failures.

In analyzing the test results reported by Kenneth C. Cox[5] in 1941 it was found that inadequate data were reported for the compressive strength of 6 by 12-in. cylinders. From the value which seemed best established at a cement-water ratio of 1.25, and from the trend of the curves of strength vs. C/W ratio, the strengths determined as representative of the concrete in the beams were the following:

C/W Ratio, by Wt. :	1.00	1.25	1.50	1.75
f'_c, psi :	1700	3100	4500	5800

A graphical comparison of theoretical and experimental resisting moments from Cox's tests is given in Fig. 12 where $M_{ult.}/(bd^2f'_c)$ is plotted against percentage of steel. In the theoretical curves, however, the effect of stress above the yield point is not shown. It is quite apparent that stresses above the yield point were developed in the beams with low percentages of steel.

In Fig. 13 the data obtained by Cox and by Slater and Lyse are plotted so as to show $M_{ult.}/(bd^2f'_c)$ vs. pf_s/f'_c where f_s is the calculated stress in the steel at failure. The test values have been divided into three groups depending on whether the calculated steel stress is less than, equal to, or greater than the yield point. Because of the assumptions which had to be made regarding the properties of the steel above the yield point, the points given for f_s greater than f_y may be disregarded. It will be noted that most test values fall within a range of ± 5 per cent of the approximate values given by the equation

$$\frac{M_{ult.}}{bd^2f'_c} = \frac{pf_s}{f'_c}\left(1 - \frac{1}{2}\frac{pf_s}{f'_c}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (11)$$

provided that f_s is determined according to the conditions stated herein.

6. INDIRECT DETERMINATION OF THE PLASTICITY RATIO FROM TESTS OF BEAMS

For the purpose of determining the plasticity ratio β indirectly from compression failures of beams, one may regard Eq. (11) as sufficiently well established that it may be used to calculate steel stresses at failure.

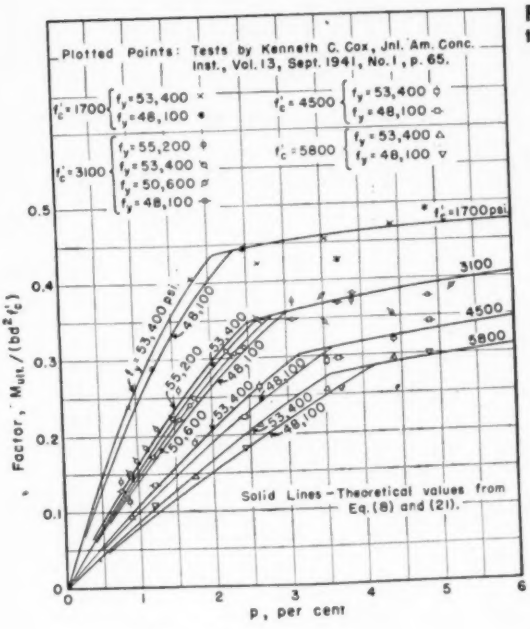

Plotted Points: Tests by Kenneth C. Cox, Jnl. Am. Conc. Inst., Vol. 13, Sept. 1941, No. 1, p. 65.
f'c = 1700 { fy = 53,400 × ; fy = 48,100 • }
f'c = 3100 { fy = 55,200 ; fy = 53,400 ; fy = 50,600 ; fy = 48,100 }
f'c = 4500 { fy = 53,400 ◇ ; fy = 48,100 ○ }
f'c = 5800 { fy = 53,400 △ ; fy = 48,100 ▽ }
* f'c = 1700 psi
3100
4500
5800
Solid Lines – Theoretical values from Eq. (8) and (21).
Factor, Mult./(bd²f'c)
p, per cent

Fig. 12 (left)—Comparison of theoretical resisting moments with experimental data

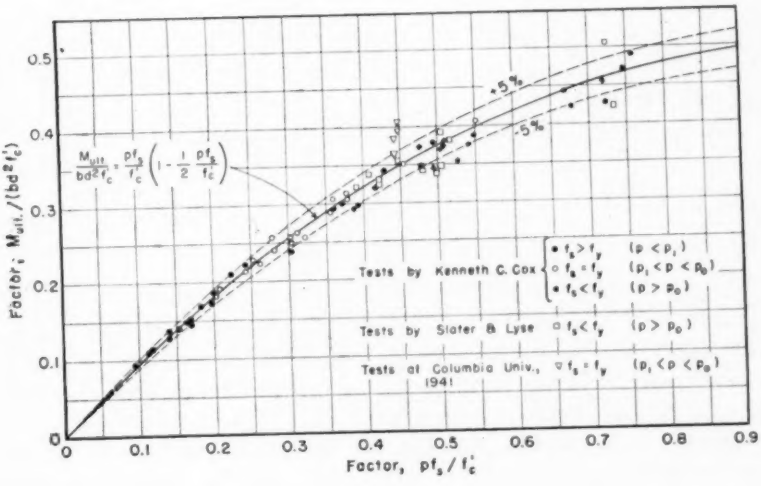

+5%
-5%
Tests by Kenneth C. Cox { • fs > fy (p < p1) ; ○ fs = fy (p1 < p < p0) ; • fs < fy (p > p0) }
Tests by Slater & Lyse □ fs < fy (p > p0)
Tests at Columbia Univ., 1941 ▽ fs = fy (p1 < p < p0)
Factor; Mult./(bd²f'c)
Factor, pfs/f'c

Fig. 13 (below)—Comparison of theoretical resisting moments with experimental data

That is, from Eq. (11)

$$\frac{pf_s}{f'_c} = 1 - \sqrt{1 - \frac{2\,M_{ult.}}{bd^2 f'_c}} = \alpha \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (26)$$

where, for convenience, pf_s/f'_c has been designated as α. However, from Eq. (14), when $\epsilon_s = f_s/E_s$,

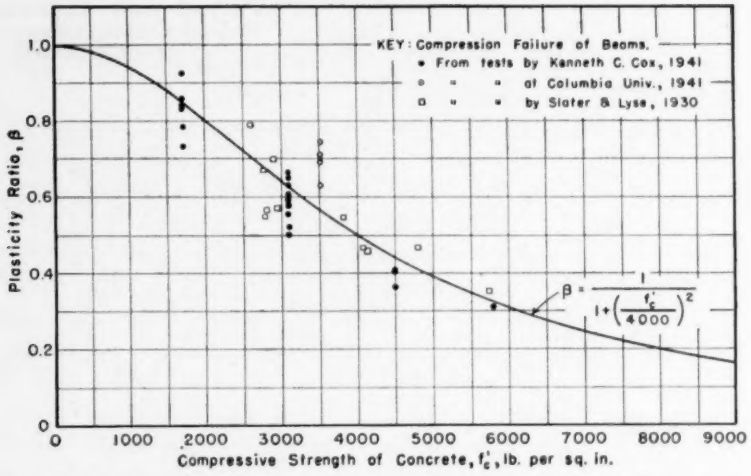


Fig. 14—Relation between β and f'_c determined from compression failures of beams

$$\alpha = \frac{\frac{1+\beta}{2}}{1+\frac{(1-\beta)\,\alpha}{np}} \qquad (27)$$

Solving (27) for β gives

$$\beta = 1 - \frac{1-\alpha}{\frac{1}{2}+\frac{\alpha^2}{np}} \qquad (28)$$

Determination of α from (26), using experimentally determined ultimate resisting moments, permits finally a determination of β by means of (28).

Determination of β from compression failures of beams, using Eq. (26) and (28), results in values represented by the plotted points in Fig. 14. A considerable spread in the data expressing a relation between β and f'_c is to be observed. This is similar to the spread which has been observed between n or E_c and f'_c and may be due to similar causes such as variability in density of aggregates, workability of concrete, relative amounts of voids, storage conditions, etc. A part of the spread is due to the impossibility of duplicating exactly all conditions which enter a test. The mean of the plotted points is seen to be represented reasonably well by Eq. (2).

7. SUMMARY AND CONCLUSIONS

A modified concept of the action of concrete in compression has been presented in this paper. According to this concept the properties which

define the stress-strain characteristics of concrete may be measured by two quantities, (1) the modular ratio, $n = E_s/E_c$, and (2) the plasticity ratio, β which is defined as the ratio of the plastic strain to the total strain of the concrete at rupture. Each of these quantities is a function of the compressive strength of concrete and, for workable concretes made with ordinary aggregates of gravel or crushed stone, may be approximated by Eq. (1) and (2).

Based on the preceding concept, an analysis has been made for the ultimate strength of rectangular beams reinforced in tension only. This analysis shows that a beam with reinforcement in excess of a critical percentage p_o, where

$$p_o = \frac{f'_c}{2f_y} \frac{1+\beta}{1+\frac{(1-\beta)\,f_y}{nf'_c}} \quad \ldots\ldots (15)$$

will fail by compression. A beam with reinforcement less than a second critical percentage p_1, where

$$p_1 = \frac{f'_c}{2f_y} \frac{1+\beta}{1+\frac{(1-\beta)\,E_c\epsilon'_y}{f'_c}} \quad \ldots\ldots (19)$$

will fail by tension in the steel but will develop a steel stress in excess of the yield point. A beam with reinforcement intermediate between p_1 and p_o will fail by tension in the steel and at ultimate load the steel stress will be the yield point. A satisfactory determination of the ultimate resisting moment may be made from the equation

$$\frac{M_{ult.}}{bd^2f'_c} = \frac{pf_s}{f'_c}\left(1 - \frac{1}{2}\frac{pf_s}{f'_c}\right) \quad \ldots\ldots (11)$$

wherein: f_s is given by Eq. (19) for $p > p_o$; $f_s = f_y$ for $p_o > p > p_1$; f_s is given by Eq. (24) for $p < p_1$. The relative depth to the neutral axis, just prior to rupture of the beam, is given by Eq. (7).

Tests show reasonable agreement with the analysis.

An indirect determination of the plasticity ratio, β, from tests of beams has shown that the variation of β with f'_c is given approximately by an equation of the form

$$\beta = \frac{1}{1+\left(\frac{f'_c}{4000}\right)^2} \quad \ldots\ldots (2)$$

wherein f'_c is the numerical value of the compressive strength of concrete when expressed in psi.

8. ACKNOWLEDGMENT

The study reported in this paper was made as a part of an investigation of the effect of concentrated loads on reinforced concrete bridge slabs being conducted in the Engineering Experiment Station in cooperation with the Public Roads Administration of the Federal Works Agency and the Illinois Division of Highways. The project is under the administrative direction of Dean M. L. Enger, Director of the Engineering Experiment Station; Prof. W. C. Huntington, Head of the Department of Civil Engineering; and Prof. F. B. Seely, Head of the Department of Theoretical and Applied Mechanics. F. E. Richart, Research Professor of Engineering Materials, gives general supervision to the work of the investigation.

An Advisory Committee consisting of representatives of the three cooperating agencies is in general charge of the program of the investigation.

Particular acknowledgment is made to Professors F. E. Richart and N. M. Newmark for their encouragement and for their helpful suggestions which have been generously given throughout this study.

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Discussion of this paper should be submitted in triplicate to the ACI Secretary before September 1, 1943, for publication in the JOURNAL Supplement concluding Proceedings V. 39.

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Discussion of a paper by V. P. Jensen:

The Plasticity Ratio of Concrete and Its Effect on the Ultimate Strength of Beams*

DISCUSSION BY DOUGLAS E. PARSONS, CHARLES S. WHITNEY, H. J. GILKEY, K. HAJNAL-KONYI, PAUL W. ABELES and the AUTHOR

By DOUGLAS E. PARSONS†

The paper describes a skilful analysis of data on the flexural strength of reinforced concrete beams. Equations are given for predicting the manner of failure and for estimating the maximum resisting moments of beams that fail by tension in the reinforcement or compression in the concrete. The equations are not intended for use in routine design, and the author wisely cautions against placing too much emphasis on the ultimate flexural resistance of beams. He also calls attention to the large deformations and deflections that accompany failures in bending. It may be added that wide cracks appear before such failures, especially when the failure is by tension in the reinforcement. The paper does not present evidence justifying a general increase in design stresses.

The discrepancies between predictions based upon direct applications of the straight-line formulas for flexural stresses in reinforced concrete beams have been discussed by many writers. Methods for estimating their magnitudes were described in Institute papers many years ago by Slater et al.[7][8] The author cites references to more recent studies made with the objective of developing generally applicable formulas for predicting flexural strengths, based upon idealized stress-strain relations.

As stated by the author, data on the stress-strain relations for concrete at stresses near the maximum are very meager. Data examined by the writer do not support the correctness of the relation between the maximum deformation at failure and the compressive strength of concrete expressed by equation (3), and they are not sufficiently extensive nor concordant

*ACI JOURNAL, June 1943; *Proceedings* V. 39, p. 565.

†Chief, Masonry Construction Section, National Bureau of Standards, Washington, D. C.

[7] [8] All references, including the author's original list will be found at the end of this series of discussions.

to prove that the relations illustrated in Fig. 3 are not fairly good approximations of the actual ones. However, the value of the paper does not depend upon verification of the accuracy of the assumed forms of stress-strain relations for concrete and reinforcement. Regardless of what is indicated in this respect by future studies, the paper provides an explanation of the mechanism of failure of flexural members and a means for predicting both the type of failure and strength. For these objectives it matters but little whether or not the actual stress-strain relations are as assumed in the paper and illustrated in Fig. 3, 4 and 5. The values of β, E_c, etc., must be evaluated empirically in any case.

By CHARLES S. WHITNEY*

In the author's complete report[9] he makes the following statement which was evidently omitted in condensing the material for publication in the Journal: "It is believed that the theory presented herein will give the research worker a better understanding of the fundamental behavior of the reinforced concrete beam. It does not follow that any of the formulas given herein should necessarily be used in design. It is recognized that simple and safe approximations may properly be used as design formulas."

The paper is of considerable interest and value from a theoretical point of view but the method presented is hardly suitable for practical design. While the combination of an elasticity ratio and a plasticity ratio is theoretically attractive and logical, it is questionable whether the resulting complication is warranted by the existing test data. Actually both of these ratios are highly variable and are effected by many factors, so that the joint use of two inaccurate empirical values cannot be expected to result in great accuracy.

As the result of an extensive series of tests, Brandtzaeg[10] published a similar method in 1935 in which he proposed the use of a plasticity ratio and a parabolic area in place of the triangular portion of the stress block. Theoretically and without complicating the mathematical treatment, this should give closer results than the use of the triangle with a slope equal to the initial modulus of elasticity.

The writer also believes that the author's assumed stress-strain curves shown in Fig. 3 can be improved both in the ascending and the descending portions. There is some evidence [11] that in general the maximum load on a concrete cylinder occurs at a greater strain for the stronger concretes than for the weaker. This is born out by the author's curves in Fig. 6 although the curves are not completely shown. The unreliability of any assumed stress-strain curve must however be kept in mind. As pointed

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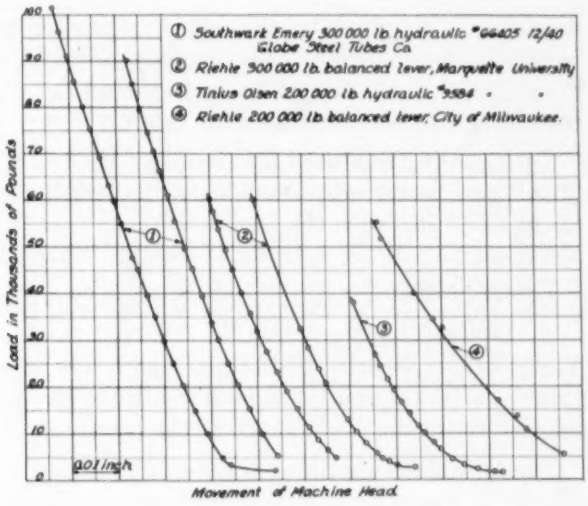

Fig. A.—Elastic movement of testing machine head on release of load

out by Gilkey and Murphy[11] in many cases the curves for high and low strength concretes are very similar in shape while in others the higher strength curves are straighter than the lower strength curves.

There is no apparent reason why the ultimate strain ϵ, shown in Fig. 7 should have such low values. Saliger and Brandtzaeg have reported maximum concrete strains in beams approximately twice as great as those assumed by the author for concrete strengths up to about 3500 psi.

The vertical descent of the stress-strain curves shown in Fig. 7 is without basis and is evidently the result of a prevalent idea which confuses time with space. High strength concrete cylinders fail suddenly but as a matter of fact during the failure the elastic movement of the testing machine head imposes a large additional strain on the concrete. This strain may be considerably greater than the total strain up to the time that failure starts. At that time a large amount of elastic energy is stored in the cylinder and in the machine (about three times as much in the latter as in the former) and the release of this energy causes the breakdown of the cylinder.

Because no information on the elastic characteristics of testing machines was available, the writer has just made tests on four machines of 200,000 and 300,000 lb. capacity which are probably typical of those used for concrete cylinders. One was a new Southwark Emery hydraulic 300,000 lb. machine while the others were different kinds as listed in Fig. A. There was a remarkable similarity in the behavior of the four machines.

The test was made by placing a hydraulic jack under load in the machine and then releasing the load on the jack without operating the

machine. This simulates the condition existing when a concrete cylinder breaks in the machine. The distance between the machine heads was measured at each decrement in jack load. The observed movement is plotted against the total load in Fig. A.

Disregarding the rapid movement of the heads as the machine loosens up at very low loads, the 300,000 lb. machines show a movement of about 0.038 in. when the load is released from 100,000 lb. to zero. For a standard 6 x 12-in. cylinder of 4000 psi concrete this is about

$$\frac{28.3 \times 4000}{100{,}000} \times 0.038 \times \frac{1}{12} = 0.0036 \text{ in. per in.}$$

The elastic movement of the machine heads during the breaking of the cylinder after the maximum load is in this case about twice as great as the total strain in the concrete up to the maximum load.

The movement shown by the 200,000 lb. Riehle machine is still greater. (Curve 4, Fig. A). The apparent stiffness of the 200,000 lb. hydraulic machine may have been due to leakage during the test.

Fig. B shows typical stress-strain curves for three grades of concrete combined with the load-strain curve for the testing machine. The latter curve has been reduced to the same terms as the concrete curve by dividing the measured values of machine load by the area of the cylinder and dividing the movement of the head by the length of the cylinder. These curves explain why the higher strength concrete fails suddenly.

As load is placed on the cylinder, a plastic strain and an elastic strain are produced in the concrete while the machine is also strained elastically. The energy applied to the machine is partly used up in deforming the concrete plastically and the balance except for friction losses is stored in the concrete and in the machine as potential energy. While the load is building up, the speed of loading will effect the amount of plastic flow and the shape of the stress-strain curve. It will not materially effect the amount of potential energy built up in the concrete and the machine due to elastic deformation.

When the maximum load is reached, the plastic deformation of the concrete which has been increasing, becomes more rapid and the machine starts to transfer its potential energy to the cylinder. Shortly after the maximum load, the slope of the concrete curve becomes equal to that of the machine curve. The elastic recovery of the machine at that stage is rapid enough to maintain the load required to continue the straining of the cylinder without operating the machine. At this point the strain starts to increase automatically and rapid failure follows.

During this failure with the machine not operating, the head follows the load-strain curve marked C in Fig. B, and the true stress-strain curve

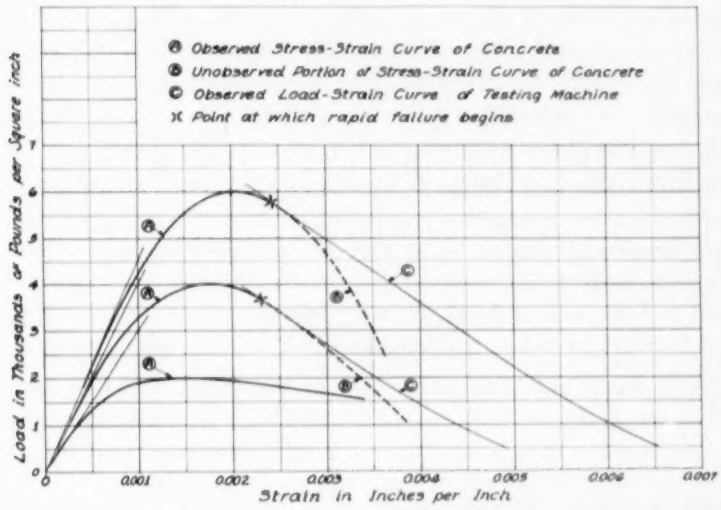

Fig. B—Effect of the elasticity of testing machine on the failure of concrete cylinders

of the concrete lies below C. The location of the true stress-strain curve has never been determined, but it could not be vertical because a failure could not occur without an increase in strain. Increasing the rigidity of the machine would defer the sudden failure and permit measuring more of the stress-strain curves.

With high strength concrete and a perfectly rigid machine, it is probable that sufficient energy would be stored in the cylinder itself to cause sudden failure at some strain. This strain might approximate the strain observed in beams at maximum load which are ordinarily two or three times as great as the strain in cylinders at maximum load.

Fig. B illustrates why 2000 psi concrete does not fail suddenly. The stress-strain curve never becomes as steep as the machine load-strain curve at any load and a gradual failure results. There is however no doubt but that a sudden failure could be produced with a very flexible machine.

Based on the curves of Fig. B, the approximate amount of energy applied in loading cylinders of different strengths up to the point of maximum load has been calculated to be as follows:

Approximate Applied Energy in Inch Pounds

Strength of Concrete (psi)	2000	4000	6000
Lost in plastic deformation of concrete	437	840	1210
Stored in concrete through elastic deformation	227	680	1360
Stored in machine through elastic deformation	618	2140	4530
Total applied	1282	3660	7100
Total potential energy in concrete and machine	845	2820	5890

This indicates that the amount of energy lost in deforming the concrete plastically is roughly proportional to its strength while the elastic energy stored in the concrete and the machine increases much more rapidly than the strength. At maximum load, the machine potential energy is about three times that in the concrete.

No definite data are available regarding the amount of energy required to produce failure after the maximum load is reached; but since the ultimate strains appear to be smaller for stronger concrete, the energy required for destruction probably does not increase as rapidly as the strength. This indicates again why the strong cylinders appear to destroy themselves near maximum load while the weaker ones do not.

The writer will not attempt to discuss the plasticity ratio method in detail but he thinks that it should be based on a better approximation of the actual stress-strain behavior if it is to satisfy the needs of research workers and justify the extra elaboration due to the use of both the elasticity and plasticity ratios.

Since both these ratios are empirical functions of the concrete strength, they can both be eliminated for practical design purposes and simple formulas can be written involving only the concrete strength and the steel strength in addition to the necessary dimensional quantities. It should be possible to develop the very much simpler plastic theory* so that it will be adequate for design. Research directed along that particular line is greatly needed and the writer hopes that the American Concrete Institute will sponsor such a project as part of its postwar program.

By H. J. GILKEY†

INTRODUCTORY REMARKS

In his introduction the author states, "In this paper there is described a concept of the action of concrete in compression which permits the derivation of formulas for the ultimate resistance of beams." The paper and its companion bulletin (9) are frankly efforts to explain in mathematical vernacular the oft-observed fact that reinforced concrete beams always develop a greater ultimate bending resistance than that calculated by the conventional elastic theory and assumptions of design. As such, they are sequels to, and an extension of, a theory of flexural analysis which is of European origin (Saliger and others, see author's references No. 1, 2, 3) and which has been actively promoted in this country by C. S. Whitney (references 4 and 4a). The approach is similar to Whitney's

*See reference 4a and Mr. Whitney's discussions of references 5 and 12, p. 584 - 25.

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and is virtually identical with Saliger's up to the introduction of the "plasticity ratio." As implied by the title, it is the plasticity ratio concept and use upon which the justification for the paper is apparently based.

It is with the utmost reluctance that the writer undertakes to criticize this paper adversely in several important respects. He makes the following allegations:

(a) The paper is speculative in its background assumptions and is not the rigorous analysis one has a right to expect from such a pretentious and involved treatment. Some of the implications are highly misleading.

(b) The paper is apparently incomplete; there has evidently been considerable "behind-the-scenes" manipulation, the results from which the reader is expected to accept on faith; results which cannot be verified by anything to be found within the paper.

(c) The paper purports to establish a linkage between the *compressive* stress-strain diagram for concrete and the behavior of concrete beams, thereby augmenting and perpetuating a false concept of concrete compressive stress-strain behavior which has been widely circulated and accepted through the work of Saliger.[1]

Before starting the general discussion of contentions (a), (b) and (c) it may be well to call attention to an evident discrepancy in the definitions given for "plasticity ratio." In the synopsis on p. 565 it is defined as, "the ratio of the plastic strain to the total strain at rupture of the concrete." On p. 566, "A measure of the extent of the plastic action is given by the ratio of the plastic to the elastic strain and is called the 'plasticity ratio'." These two definitions appear to be mutually incompatible. According to Fig. 3(b) the former definition seems to be the favored one.

DISCUSSION OF THE CRITICISMS OFFERED

In studying this paper one intuitively assumes that Fig. 1, 2 and 3 are based on a solid background of carefully generalized data from actual compressive tests and that such figures as 11, 12 and 13 constitute the "proof of the pudding;" that they are really final checks against independent tests of beams which demonstrate the overall adequacy of the analysis, proving, among other things, that the slight liberties taken, in squaring off the mosque-like figures of 3(a) to construct the near-gothic forms of 3(b), were negligible in their effect on the accuracy of the treatment.

Evidently, this is not the case; Fig. 1 is apparently the only figure in the paper that so much as makes a pretense to being based on compressive

tests. Incidentally, even Fig. 1 is formalized and somewhat over-idealized. As the writer has pointed out elsewhere, (ref. 13, p. 10), the strains at the ultimate have no such definite values as Fig. 1 appears to show. Generally speaking, the writer's observations indicate that instead of all comparable concretes reaching their ultimate loads at identical strains, the stronger concretes seem to have somewhat greater strains at the ultimate than do the weaker ones. Moreover, referring to the last sentence on p. 566, the author might be surprised to note how nearly identical the shapes of curves may often be for concretes of widely varying strengths (ref. 13, p. 27-29). Suffice it to say that Fig. 1 isn't a bad representation, in the writer's estimation, and that there are at hand much more important items than that of quibbling over any possible minor discrepancies there.

Fig. 2 is, as the author has evidently recognized, nothing more than Saliger's concept of a type of compressive stress-strain diagram which would, if true, explain in terms of the compressive stress-strain diagram, the excess strength of reinforced concrete beams. That the author accepts Fig. 2 as nothing more than a hypothetical concoction seems reasonably clear from the legend, "Possible stress-strain diagram for concrete at strains approaching rupture—Saliger." As an additional indication that the author was fully alive to the weakness of what purports to be his take-off, one may quote his own words on p. 567 where he discusses strains beyond the ultimate, "The exact character of this portion of the stress-strain curve for concrete of any strength is not well established. In fact, test data are very meager and are likely to be greatly influenced by the speed of testing."

For an investigator thus clearsightedly to post his own stop light and then ride brazenly through it with an elaborate development, the validity of which rests squarely on the validity of his premise is a thing scarcely to be condoned in a scientist, even though the writer does feel that, in fairness to the author, there are in the present instance certain extenuating factors which should not be overlooked.

The Saliger hypothesis had been advanced and published by a widely recognized authority and its validity had not been seriously challenged during the several-year interim; moreover, it has seemed to square unusually well with the available data from flexural tests. While not having been verified experimentally from the bottom up, it has appeared to have excellent verification from the top down.

Although the writer is convinced that the Saliger diagram is emphatically not representative of the behavior of any concrete under axial compression, he agrees that there is every reason to believe that it con-

stitutes an excellent qualitative portrayal of the behavior of the concrete on the compressive side of a beam. In other words, the writer believes that Saliger and his successors have simply lost sight, for the moment, of the fundamental difference between the respective situations in a uniformly axially loaded member and a member in flexure where all fibers are not equally stressed and there are opportunities for redistribution. If, for example, the author's caption for Fig. 2 had been, "Possible (or even probable) stress-strain diagram for some of the more heavily stressed fibers of a beam at loads approaching the ultimate resistance of the member," the author's take-off, while still speculative, would have been much less vulnerable.

Much of Saliger's backlog of evidence, such as it is, seems to date back to relatively early tests when techniques for controlling rate of loading and for securing good observations on strains were in an elemental state. Even today it would require some of the best of modern testing and observational equipment and carefully planned, skillfully executed tests to secure significant evidence on the post-ultimate compressive behavior of concrete. It is not surprising, therefore, that Saliger should have worked backward from the beam-test strength results to evolve an imaginary compressive stress-strain diagram which would account for the observed excess beam strengths. He failed to take tangible cognizance, however, of the fact that the stress-strain diagrams for flexure and compression are bound to differ in certain vitally fundamental respects; he over-shot his mark badly in attempting to carry his explanation back to the compressive stress-strain behavior, as have his successors and disciples.

In making the transition from Saliger (Fig. 2) to the representations of Fig. 3 the author states on p. 567, "For the purpose of explaining the behavior of concrete in a flexural member, a typical set of stress-strain curves extended to rupture for gravel concrete is assumed as shown in Fig. 3a." Here at the most vital juncture in the whole development the technical trail has become very, very dim. This set of curves is supposed to be *typical* of what? A reader would properly expect them to be the generalized results of experiments which are themselves readily available for checking the reasonableness of the generalization. If compressive tests are not their origin (and apparently, for the extensions beyond the ultimate, they are not) then certainly their source and derivation should be carefully and fully indicated, for, after all, what is the significance of any subsequent development that might be based on such an unusual set of diagrams of mysterious and wholly unexplained origin? After having previously indicated the dearth of evidence on the compressive stress-strain behavior beyond the ultimate, on just what factual basis is

the author able to show such a set as typical? How did he decide just how far in terms of strain, each strength of concrete was able to stand up under it's ultimate stress?

Apparently, the "typical" post-ultimate compressive stress-strain diagrams of Fig. 3 never saw a compressive specimen but have been guessed in purely by a trial and error technique, not by working forward from Saliger (Fig. 2), but by working backward from the check series of Fig. 11, 12, 13 and 14. In other words, Fig. 3, instead of being the take-off, really appears to be the end point of the analysis. The real sequence in the preparation of the paper appears to have been (a) the qualitative acceptance of Saliger's premise followed by (b) a working backward from the composite data of Cox, Columbia University and Slater and Lyse to evolve by trial a set of fictitious "typical" compressive stress-strain diagrams resulting in the architectural effects of Fig. 3.

If the foregoing is correct, what appear to be such excellent checks are not checks at all since the nicely plotted points on Fig. 11, 12, 13 and 14 represent nothing more than the closure of a manipulative circular sequence of operations; a return to the starting point. This is not to imply that a reasonable check with results from good independent series of beam tests is not to be expected; it is merely to point out that such a check is not a part or parcel of this analysis.

To conduct extensive analyses from other investigators' data is always a difficult and somewhat hazardous undertaking but in a project which is already such a heterogeneous mixture of the speculative, the empirical, the mathematical and the mystical, it is doubtful if the ingenious device of supplying hypothetical strengths on the basis of the recorded cement-water ratios (p. 578) has weakened the development in any essential respect (in spite of the great range of cement-water ratio vs. strength relationships that exist for different cements and mixtures.

COMPRESSIVE VS. FLEXURAL STRESS-STRAIN BEHAVIOR

Notwithstanding the keen insight displayed by Saliger and his successors with respect to what occurs within a stressed beam as the proportional limit of some fibers is exceeded and/or ultimate stresses are approached, or even passed, all of the writers seem to have displayed a common weakness in their attempted linkage of compression and flexural phenomena. They have seemingly failed to take cognizance of the fact that in the axially uniformly loaded member, "as goes the fiber, so goes the member"; when one fiber reaches its maximum resistance, all do; the failure of the member is coincident with the failure of a fiber.

In the case of the flexural loading of a member having a plastic range of stress, there are reserves that are brought into action as needed. An

overloaded fiber can deform plastically still offering substantial resistance while other lightly-stressed fibers are brought into service to take on increased strains and correspondingly increased stresses. The ultimate resistance developed by the member is not developed simultaneously with the ultimate resistance of some fibers. Various readjustments, coalitions of over and under-stressed fibers which add to the sum total resistance of the member, continue to occur long after some parts of the cross-section have reached or even passed the peak of their respective resistances.

The redistribution within the beam and the probable manner in which it accounts for the reserve strength found in beams is discussed, illustrated and dwelled upon by Saliger[1], Whitney[4 and 4a] and others. Some of the elementary textbooks on strength of materials show qualitatively the similar action which takes place in plain concrete, stone or cast iron flexural members to give them moduli of rupture as much as twice the ultimate tensile strengths of the materials.

It needs to be continuously recognized and realized, of course, that there is no magic in these apparent differences between flexural and axial strengths. The redistribution that follows the non-proportional action just discussed simply produces departures from what the conventional straight line formulas of flexure assume to be happening, giving apparent rather than true values for computed ultimate stresses. That the well-known hyper-elastic compressive behavior of materials in flexure should have been carried over to the case of axial compression is obviously due solely to lack of thought rather than lack of understanding. Analogy, valuable tool that it is, can be, and often is, blindly misapplied by the best of us.

At first thought one might raise the question, "Why the urge to explain flexural behavior in terms of the compressive stress-strain diagram anyhow?" The answer appears to be two-fold and rather simple. First, the compressive stress-strain diagram as determined by the regular short-time test is the recognized medium for evaluating the structural properties of concrete; it supplies the values used to describe the elastic and ultimate strengths, the stiffness, elasticity, extensibility, resilience and toughness of the concrete. Flexural as well as compressive, bond and diagonal tension design stresses are all expressed as percentages of the compressive strength as determined by the conventional test cylinder and established test procedure. Second, to determine what the actual stress above the proportional limit may be at any specific part of the flexural cross-section is not a simple matter. It does seem strange that with a definitely computable bending moment at any section, it should not be possible to determine readily and simply just how the total compressive and tensile

stresses (equal to one another as a basic fact of statics) are distributed over their respective portions of the cross-section. The problem of breaking down the known value of the bending moment at any section into the correct resultant and/or distributed forces and arm of the resisting couple is in its difficulty and uncertainty, not unlike that of attempting to reproduce the force system within the wall, or other support of a cantilever beam (or for any other indeterminate stress situation). The fact that in such situations the product of the resultant stresses and their arm is definite and known doesn't shed much light on the questions of distribution and redistribution of stress within the beam or within the wall.

With this much for general comment, it is now desirable that the compressive test phenomena be scrutinized in some detail to see why the Saliger type of short-time compressive stress-strain diagram can scarcely be accepted as a correct portrayal of what occurs.

COMPRESSIVE TESTS

On the basis of a great number and variety of concrete compressive tests in which strains were observed and recorded all the way to the ultimate load the writer feels qualified to speak with some assurance on compressive testing. Just prior to the ultimate, when the load is practically at a standstill, the strain continues to increase with the travel of the testing head. As the ultimate is attained and passed, the load drops off rapidly for a time while strains increase whether the testing head is held stationary or is allowed to continue the gradual rate of descent at which the ultimate load was reached. With the head in motion the compressometer dial continues to record rapid increase of strain under rapidly decreasing load followed by almost immediate collapse of the specimen. There is no such bearing up under a load approximating the maximum as is shown in either Fig. 2 or Fig. 3.

In Fig. 1 of Whitney's Am. Soc. C. E. paper (ref. 4a, p. 254) from the bachelor's thesis of Kiendl and Maldari[14]) are shown some actual data, of sorts, on stress-strain relationships beyond the ultimate. These are doubtless correct portrayals of what occurred in spite of the fact that they are, "highly unusual stress-strain curves for concrete cylinders" as is pointed out by Hadley (ref. 15, p. 292). It is remarkable that with a continuously descending testing machine head the specimens should have held together.

These observations were on 28-day standard cured 6 by 12 in. specimens tested in a Riehle 200,000 lb. (presumably lever type) testing machine with the head traveling continuously at the (idling) speed of 0.06 in. per min. (Actual speed under load, undoubtedly less than 0.06 in.

per min.). Readings were continued to the stage where damage to the specimens precluded further observations.

Granting the validity of the data, the writer fails to see that they have any significance as an index to the supporting strength of an axially loaded member. The member has passed its ultimate resistance and if the head of the testing machine had not been restrained from following down, the specimen would have collapsed the instant the ultimate load was passed.

By reversing the testing machine as soon as the ultimate load is reached the writer has found by test that a compressive specimen can often be reloaded to a new ultimate as high as 90 per cent of the original ultimate. As a practical matter, however, that 90 per cent just wouldn't be enough to support the 100 per cent load that is presumably resting on the specimen. Even a 99 per cent residual resistance isn't enough. (By way of information, it may be stated that just a few successive reloadings to successive new—but progressively lower—ultimates, results in complete disintegration but this is beside the present point.) The point being made here is that, in terms of an axial load actually resting on a member, effective resistance ceases the instant the ultimate is passed regardless of whether that ultimate be the ultimate for a cylinder, the somewhat higher ultimate for a cube or the lower ultimate for a slowly conducted test. Just how hard the failing specimen happens to be pushing upward against the descending load doesn't matter if the upward push is less than 100 per cent of the propelling downward push.

Once the ultimate is passed, the axially loaded member is on its way out (and fast, if the load is free to follow down as for a load resting or hanging on a member). Whether the specimen accepts its ultimate fate (collapse) cheerfully, offering only token resistance or whether it remonstrates and drags its figurative feet every fraction of the way is beside the point; it is doomed from the instant the downward push exceeds the magnitude of the greatest resistance the specimen can muster. Resistance can cushion the fall but it won't prevent it. Even a freely falling body will exert an upward push against another descending body above it if the acceleration of the latter exceeds the acceleration caused by gravity. The Kiendl-Maldari data which, even for the conditions under which they were secured, supply little support for the Saliger assumption, are entirely meaningless with regard to having any significance with respect to the phenomena under discussion here.

Rather than striving to endow a uniformly loaded compressive cross-section with some fantastic super characteristic whereby a 75, 80 or 95 per cent upward push halts in midair a 100 per cent downward load, might

it not be more rational to start from the premise that beams are distinctive because they are beams, endowed as all plastic beams are endowed with a capacity for stress redistribution?

This is, of course, exactly where the author and his predecessors have landed as the second major step in their analyses. Why not let it go at that? Why not drop the erroneous pretense that Fig. 2 and Fig. 3 are valid representations of the stress-strain behavior of concrete in axial compression? Use the equivalent of Saliger's assumption, if desired, but with some such legend as, "Possible stress-strain diagram for some of the more heavily stressed fibers of a beam at loads approaching the ultimate resistance of the member."

CONCLUDING COMMENTS

In spite of the rather vigorous exceptions taken to certain primary aspects of this paper, it should be clearly noted that the scope of the criticism is distinctly limited.

The excess strength of beams over that given by the conventional straight-line analysis is an observed non-controversial fact, recognized many years ago even before the general adoption of straight-line in preference to parabolic or other type of design formula. Any valid analysis that places the extent of the excess strength to be expected in the easily computable category is all to the good and the objective sought by Saliger, Whitney, the author, et al is certainly to be commended rather than criticized.

Exception is not taken to the general objective; nor does the discussion concern itself with the correctness of the end-point reached by any of these workers, or the relative merits or possibilities of usefulness of the plasticity ratio approach in comparison with the preceding contributions; moreover, the writer is not taking sides on whether or not the established straight-line procedure should be discarded completely, in part, or retained. (The author (p. 565) likewise professes neutrality on this point.)

Because the observed excess strength of flexural concrete members is inherent in the compressive behavior of concrete in a beam, it has been an inevitable sequel of established methods that our conventional "balanced" reinforced beams are never balanced; that they are always disproportionately strong in compression. In the interest of economy (in normal times when steel is not a war scarcity) and of common sense, that discrepancy needs to be corrected. To remedy much of the current *unbalance* of "balanced design" one doesn't have to discard the straight-line design technique. As pointed out by Hadley (ref. 15, p. 294), simply increasing the permissible concrete compressive stress used in flexural design would go far toward rationalizing current practice without any

upsetting of established processes. Whether the added precision potentially possible via the Whitney or Jensen route is enough to be a controlling factor is a question to be settled in the light of comparative studies and the collective judgments of specification-making groups.

The objections offered to this paper are not that its aims are at fault or that its objectives are unworthy; in a sense the right thing is simply believed to be being done in a wrong way. As indicated at the beginning; the premise is alleged to be unsound and the treatment is accused of being devious and mystical. Moreover, like its predecessors in the field, what is believed to be an entirely erroneous concept of standard stress-strain behavior is accepted and heavily built upon. Such faults are inexcusable in an engineering treatise, no matter how commendable the aim or how nearly correct the ultimate answer may be.

By deleting all allusions to compressive stress-strain behavior (except in beams), by indicating clearly that representations of the compressive diagrams of Fig. 3 are wholly empirical and apply only to the compressive action that *may* exist within a beam, and that the exhibits of Fig. 3 are in essence the end point of the paper rather than the beginning of an analysis, the writer believes that the author could greatly strengthen his paper and make it something of an asset rather than a distinct liability to the advancement of concrete understanding.

Perhaps some of the criticisms are unwarranted and perhaps others are unduly severe; in some respects the writer may be entirely at fault through his own ignorance and failure or inability to understand. The closure can be relied upon to set the record straight with respect to all such. The writer hopes, however, that in the preparation of his closure the author will not restrict his efforts to a mere attempt at vindication. Right or wrong, the author's path has in places been devious and difficult to follow; undoubtedly some of the difficulties encountered by the writer are representative of points that have puzzled and will continue to puzzle others; points that need to be cleared up rather than waved aside or covered up. Some of the questions raised call for answers, real answers and explanations; possibly some admissions. The writer hopes that the closure will be made to supplement both the paper and the discussion in such a manner as to clarify where haze obscures and correct where error exists, let face be lost where it may.

By K. HAJNAL-KONYI*

Mr. Jensen has made a very valuable contribution to the theory of reinforced concrete beams. It has been known for some time that the

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value of $\frac{M_{ult}}{bd^2f'_c}$ in over-reinforced beams is not constant but increases with decreasing f'_c. This was especially pointed out by R. Maillart[16]. Mr. Jensen's paper explains this fact and gives a closer approximation of the ultimate bending moment of over-reinforced beams than the co-efficients suggested by Saliger[1], Whitney[4a] and Cox.[5]

In regard to beams with a percentage lower than p_i (which form the great majority in actual practice) the writer would like to draw attention to a factor not mentioned in Mr. Jensen's paper: the destruction of bond between steel and concrete within the range of maximum bending moments.

For a beam with constant $maxM$ over a substantial part of the span (and only such beams are reliable for the analysis of ultimate stresses) Mr. Jensen's Fig. 5 (left) is correct if the bond between steel and concrete has been destroyed over the full length where maximum M occurs. This happens with steel having a definite yield point. If, however, the bond between the cracks is not destroyed, ϵ_s in Fig. 5 (left) should denote the *average* steel strain. The actual strain is variable. Within the cracks and in their immediate vicinity it is greater, between the cracks (where the concrete still acts in tension) it is considerably smaller than the average. This occurs for instance with work-hardened steel, which has no definite yield point.

The writer has conducted tests on beams reinforced with square twisted bars[17]. With the exception of very slender beams the steel was fractured in all beams with percentages not exceeding .9 per cent. This percentage is near the limit up to which the steel can be used in high grade concrete at a permissible stress of 27,000 psi without steel in compression, according to present regulations.

The minimum ultimate elongation of the bars used in these tests, as obtained from standard tests, was 12.4 per cent and their ultimate strength was 81,600 psi.

This type of failure cannot be explained with the diagrams and formulas in Mr. Jensen's paper. Fracture of the steel is only possible if the strain of the steel is variable within the range of $maxM$ and the maximum strain only occurs over very short lengths.

The stresses f_s in Mr. Jensen's formulas and diagrams are minimum values, based on the assumption of a uniform strain within the whole range of $maxM$. If the bond is not destroyed over the full length of this range, ϵ_s is variable and f_s reaches higher values than corresponding to the average ϵ_s.

The writer's tests were confined to percentages not exceeding 1.19 per cent and the bars were 5g (.212-in.) to ½-in. With increasing size

of bars, all other conditions being equal, it is more likely that the bond is destroyed within the whole range of $maxM$ in which case the bars cannot be fractured when the beams fail and Mr. Jensen's formulas become applicable.

In general, the ultimate stress of steel at failure if $p < p_1$ depends not only on the characteristics contained in Mr. Jensen's formulas, but also on the size of the bars and on the nature of their surface. The behavior of steel with a definite yield point is quite different from that of steel without yield point, especially of work hardened steel. This difference is not sufficiently manifested in the stress-strain diagrams. For square twisted bars not exceeding ½-in. the ultimate stress at failure of the beam is in all practical cases equal to or even in excess of the ultimate strength of the bars as obtained from standard tests.

The limiting size may be even higher than ½-in. as would appear from tests made by Johnston and Cox[12]. The following table contains the results of 4 of their beams. It combines data from their Tables 1 and 2 of the article referred to. Out of the 8 beams reinforced with square twisted bars, 4 failed in diagonal tension and had, therefore, to be excluded from the following comparison.

Beam No.	Number and Size of Bars	p %	Ultimate Load: Structural Yield Point Load	Ultimate Strength: Yield Point
T-1-II	5½-in. sq. tw.	.87	$\frac{54250}{45200} = 1.20$	$\frac{70000}{58400} = 1.20$
ST-1	2½-in. sq. tw.	.35	$\frac{23550}{18200} = 1.30$	$\frac{71300}{64500} = 1.10$
ST-2	2⅝-in. sq. tw.	.54	$\frac{34800}{28900} = 1.20$	$\frac{70000}{58200} = 1.20$
ST-3	2¾-in. sq. tw.	.78	$\frac{49400}{42400} = 1.16$	$\frac{70300}{61000} = 1.16$

In 3 beams, with p varying between .54 and .87 per cent and the size of the bars between ½-in. and ¾-in., the ratio ultimate load: structural yield point load is exactly the same as the ratio ultimate strength of bars: yield point. In the case of beam ST-1 (p = .35 per cent, ½-in bars) the first ratio is far in excess of the second.

The yield point of small size mild steel bars may also be exceeded more than one would expect from Mr. Jensen's formulas, but it would not be advisable to rely on this additional excess, since with mild steel bars in most cases bond is destroyed within the whole range of $maxM$ before the beam fails.

The writer cannot go here into further details of his tests and has to refer to his paper[17].

By PAUL W. ABELES*

Mr. Jensen's paper constitutes a very important step in the development of formulas for the design of reinforced concrete beams, based on the ultimate strength. Generally, the method of calculation and the formulas presented by Mr. Jensen represent in the writer's view a great improvement as compared with Saliger's suggestion. In the latter it was pretended to provide a theoretical method without giving any plausible explanation and it is Mr. Jensen's merit to have developed this suggestion to a method, having a theoretical foundation.

In the following, the writer would like to discuss mainly case (3) where the steel stress is higher than the yield point stress. In the writer's opinion it is not possible to present theoretical formulae for this range on a general line to cover all possibilities, since the ultimate strength of a beam depends on the properties both of the concrete and the steel, or more correctly, on the quality of the bond between steel and concrete before failure, and thus varies greatly. In order to avoid repetition, the writer would like to express his full agreement to Mr. K. Hajnal-Konyi's contribution to this discussion, the manuscript of which he had the opportunity to read. Further reference may be made to two contributions of the writer to discussions of the same subject[4a and 5].

An ultimate steel stress higher than the strength of the steel, as referred to by Mr. Hajnal-Konyi for work hardened steel cannot be met by formulas of general applicability. Similar results to those with work hardened steel were obtained for mild steel reinforcement in spun tubular beams, in which a great bond was ensured, and the concrete was of high tensile strength (See Table 13(d), beam 16b[4a]). In this instance the theoretical ultimate steel stress amounted to about 3 times the yield point stress or about twice the ultimate strength (p = 0.30 per cent). From these results it is seen that the real stresses, occurring in the cracks, must be much smaller than the theoretical values. Bearing in mind that in such cases the bond between reinforcement and concrete is destroyed in the immediate neighborhood of the cracks only, and that therefore the real steel stresses are greatly reduced between the cracks, the phenomenon of the appearance of theoretical stresses in the cracks higher than the strength can only be accounted for if redistribution of the stresses takes place according to the hypothesis offered by the writer in 1936 (disc. 4a).

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According to Fig. C, relating to small percentages of reinforcement, the concrete tensile zone co-operates apparently to full extent in spite of the discontinuity caused by the cracks. If a full bond between steel and concrete is ensured, line (a) which is parallel to the theoretical line (b) represents the test results, whereas the latter line agrees very well with the actual behavior, if the bond is totally destroyed. For practical pur-

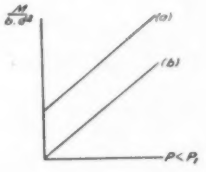

Fig. C

poses values between the lines (a) and (b) have to be considered and are dependent on the bond conditions effected by the particular concrete and steel used. In the light of these considerations it appears more correct and suitable in cases, where the bond is especially enhanced, to base the design on the real behavior of the construction, as has been suggested by the writer[5] by an extension of the Cox-formula, taking partly into account the concrete tensile zone.

However, excluding cases where an increased strength is attained due to the provision of a safe bond, it must be stated that Mr. Jensen's method appears to ensure a very good agreement between theory and practice. This can be seen from the application of his formulas (1), (2) and (14) to a test, conducted by the writer, on a prestressed, wire-reinforced beam, in which case the theoretical steel stress at failure, calculated for $n = 15$, amounts to $f_s = 252{,}000$ psi. (Cross section: 3.5 in. by 8 in.; $p = 0.229$ per cent, Mod. of El. $E_s = 26.2 \times 10^6$ psi.; concrete strength on cubes 6 in.: 6455 psi., corresponding to $f'_c = \frac{3}{4}$. 6455 = 4850 psi.).

According to formula (1) $n = 7.06$ and to formula (2) $\beta = 0.405$ resulting in the equation (14) $f_s = \dfrac{1{,}490{,}000}{1 + 455\epsilon_s}$. In the stress-strain diagram the actual elongation ϵ_s, corresponding to a stress of 252,000 psi. is 0.014, which elongation would result in a stress $f_s = 203{,}000$ psi. only; but the increase of elongations is rather rapid in this range and an elongation $\epsilon_s = 0.0115$, corresponding to a stress $f_s = 240{,}000$ psi., satisfies both the above formulas and the stress-strain diagram. In this example, relating to steel of especially high strength, a surprisingly good agreement is attained with Mr. Jensen's formulas (compare 240,000 with 252,000 psi.).

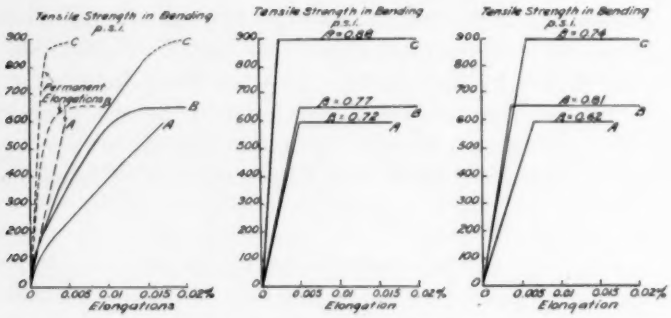

Fig. D1, D2, D3, left to right

It appears that Mr. Jensen's equations (1) and (2) represent a satisfactory approximation. On the other hand it might be somewhat difficult to derive the β values from idealized stress strain diagrams. Fig. D1 shows such diagrams for the tensile strength in bending of 3 different kinds of concrete, the permanent deformations being also indicated, taken from the writer's tests[18]. It may be mentioned that concretes A and B were of the same compressive strength whereas that of concrete C was ⅓ higher. It has to be stated that rectangular beams reinforced alternately with mild steel and high strength steel (of f_y = 87,000 to 95,000 psi.) differed only slightly with respect to the ultimate strength for the same steel and percentage of reinforcement (p = 0.38, 0.85 and 1.50 per cent) in spite of the differences of concrete strengths and plasticities. Considering the diagrams Fig. 2a it would be possible to idealize them either on the basis of the measured permanent deformations, as indicated in Fig. D2, or as suggested in Fig. D3. In each case different results would be obtained for the 3 kinds of concretes. It might therefore be better to use formula (2), instead of idealized stress-strain diagrams, for derivation of the β values.

AUTHOR'S CLOSURE

The discussors are to be thanked for their contributions of valuable supplementary data and opinions pertaining to the plastic behavior of concrete and its effect on the ultimate strength of beams.

K. Hajnal-Konyi points out the importance of characteristics of the reinforcement other than those exhibited in the standard tensile test when low percentages of reinforcement are used in beams. Undoubtedly these are important factors which, if measured quatitatively, would go far toward explaining discrepancies which exist between theoretically and experimentally determined ultimate loads for certain types of beams. It is difficult for the author to visualize the analysis as applicable to an

average condition between cracks. It is true that measured strains always represent averages over the gage length used. However, the conditions of equilibrium apply to a cut at a crack and the analysis would seem to permit actual curvatures and strains at the crack to enter, even though the curvature at an uncracked section of the beam were less. The assumption of linearity of strain distribution from top to bottom of beam undoubtedly becomes less valid near ultimate loads when the bond remains unbroken a short distance away from the cracks.

The discussion by Mr. Abeles pertains mainly to beams with percentages of reinforcement low enough to develop stresses above the yield point. There can be no doubt that the flexural strength of the plain concrete section of a beam is an important factor in calculating the ultimate strength of the beam when the total tensile resistance in flexure is of the same order of magnitude as the total tensile resistance of the reinforcement. However, this factor was not considered in the paper because of a general reluctance to consider the concrete reliable in tension. The data mentioned by Mr. Abeles are of interest. It would not seem to be particularly fruitful to attempt actual evaluations of the plasticity ratio of concrete from observed tensile strengths in bending. As discussed in the paper, the plasticity ratio is associated with the behavior of concrete in compression.

Professor Gilkey's discussion is evidently aimed not so much at this particular paper as at the whole idea of the relationship between the stress-strain diagram of concrete, including the plastic range, and the behavior of concrete in flexure. For metals, both ductile and brittle, the relationship between the stress-strain diagram and the behavior in flexure is the basis of all ordinary analyses of flexural resistance[19]. For concrete, the stress-strain diagram has been linked with the behavior of beams for approximately the same number of years as concrete has been the subject of research. In discussing beam tests[20], made in 1905, A. N. Talbot said, "Even if the straight-line relation be accepted as sufficient for use with ordinary working stresses, the parabolic or other variable relation must be used in discussing experimental data when any considerable deformation is developed in the concrete." This statement was made in direct recognition of the linkage which had long previously been established between the stress-strain diagram for a material and its behavior in a flexural member. The linkage is of course entirely due to the linearity of strain distribution in a beam even beyond the elastic range.

Contrary to Professor Gilkey's allegation (c) the paper does not purport to establish this linkage. It merely uses it in the manner in which it has been used innumerable times in the past.

With reference to Professor Gilkey's allegation (b) that the paper is incomplete and that there has evidently been considerable "behind-the-scenes" manipulation, the inference is that some damaging evidence is being withheld. This is not the case. Professor Gilkey doubtless alludes to private correspondence in which an explanation was given of the manner of first arriving at Eq. (2) and in which cylinder tests were mentioned. For the sake of the record it may be stated that the form of Eq. (2) was originally determined by trial and error using the test results obtained by Slater and Lyse[6] as an experimental basis. Later, the form of the equation was checked in the manner described in the paper. The cylinder tests referred to were merely preliminary tests made in an effort to establish a technique for determining the stress-strain diagram to rupture. Motion picture records were made in which both compressometer and dynamometer dials were photographed simultaneously. Numerous difficulties were encountered, particularly with reference to improving the visibility of the dials when they were both required to appear within each frame of the film. The technique has not been sufficiently developed to be presented to the Institute. When and if it is sufficiently developed it will be fully described. Until then it can be said that the incomplete results so far obtained are not essential to the present paper but did encourage the author in his belief in the existence of a property of concrete which may be described by a plasticity ratio.

Professor Gilkey's allegation (a) states that "The paper is speculative in its background and assumptions and is not the rigorous analysis one has the right to expect from such a pretentious and involved treatment. Some of the implications are highly misleading." The author was careful to state on p. 567 that the character of the stress-strain diagram for concrete beyond the strain at maximum resistance was not well established and that the diagrams shown in Fig. 3(a) were assumed for the purpose of explaining the behavior of concrete in a flexural member. The speculative nature of the right-hand parts of these curves is thus made clear in the paper. It is doubtless true in these, as in most simplified and idealized curves, that liberties have been taken. In particular, an unimportant trailing end on the diagram should probably flare out to the right. This would have no bearing on the maximum resistance of the beams since it would come into action only after the maximum resistance of the beam had been reached.

Professor Gilkey shows a high regard for the stress-strain diagram of concrete cylinders as such and a low regard for the relation between this diagram and the behavior of the concrete in a beam. This is puzzling since, by themselves, without connection with the structural behavior of the material, the stress-strain diagrams are practically worthless.

It should not be construed that the purpose of the paper was to establish the exact nature of the complete stress-strain diagram for concrete. The exact shape of the curves shown in Fig. 3(a) is not the end point. The purpose of the paper was to show the existence of the property of concrete described as the plasticity ratio and to show the effect of this property on the ultimate strength of beams.

The discrepancy noted by Professor Gilkey between the definition of the plasticity ratio as given in the synopsis, p. 565, and the statement given on p. 566, should be corrected in the latter place by changing the word "elastic" to "total" giving finally the correct statement, "A measure of the extent of the plastic action is given by the ratio of the plastic to the total strain and is called the 'plasticity ratio'."

The discussion of compressive vs. flexural stress-strain behavior and of the compressive test given by Professor Gilkey reveals certain misconceptions. In the first place, it should be understood that the stress-strain diagrams of Fig. 3(a) do not imply that a uniformly loaded compressive cross-section is endowed "with some fantastic super characteristic whereby a 75, 80 or 95 per cent upward push halts in midair a 100 per cent downward load" as charged by Professor Gilkey. No such characteristic is attributed to mild steel for which the load-strain curve has a well-known descending part. The testing machine is mainly a device for applying strain. The load must be measured. There is no mere weight resting on top of the specimen. Because the screws of the testing machine act as stiff springs, events may happen rapidly after the ultimate resistance is passed. Professor Gilkey's great number and variety of records "all the way to the ultimate load" are unexcelled as far as they go, but they do not go far enough to verify any of the assertions he has made with respect to the action of the concrete after the maximum resistance is passed.

Professor Gilkey states that "as the ultimate is attained and passed, the load drops off rapidly for a time while strains increase whether the testing head is held stationary or is allowed to continue the gradual rate of descent at which the ultimate load was reached." As long as the screws of the machine are stretched, it does not follow that the testing head stops moving when the screws stop turning. Consequently after the screws stop turning both load and strain may vary so that both must be observed continuously to obtain data of any significance.

As for the relation between the stress-strain diagram obtained from a compression specimen and the behavior of a flexural member, Professor Gilkey has offered no evidence that the relationship is not established through the linearity of strain distribution in the flexural member. For the determination of the ultimate resistance of a reinforced concrete

beam, there is no doubt that the effect of the tail end of the stress-strain diagram is unimportant. Conversely flexural tests to ultimate will reveal nothing of this part of the diagram.

Finally, consideration is given to the assertion that ". . . what appears to be such excellent checks are not checks at all since the nicely plotted points on Fig. 11, 12, 13 and 14 represent nothing more than the closure of a manipulative circular sequence of operations; a return to the starting point." This can only be said to be true insofar as the form of Eq. (2) is concerned. The data of Fig. 14 refute the idea that β is, for example, a constant or is unrelated to the compressive strength of concrete. No claim is made for the exactness of Eq. (2) for the same reason that it would be folly to claim exactness for Eq. (1).

The discussion by Charles S. Whitney contains interesting supplementary data. Mr. Whitney's discussion of the energy relations involved in the testing of cylinders is particularly pertinent and deserves further study. It is to be noted that the curves of elastic movement of the testing machine head after release of the load cannot by themselves explain why high strength cylinders fail suddenly. Only when these curves are used in conjunction with stress-strain curves of cylinders, which themselves show a greater tendency for plastic deformation of the weaker concretes, do they indicate when sudden failure is to be expected.

As noted previously, the vertical drop in the assumed stress-strain curves was not due to a confusion of time with space but was due to a recognition of the uselessness of the trailing end of the curve when determining the resistance of flexural members.

For comparison with Mr. Whitney's values of energy stored in cylinders at maximum load, the following calculated values of energy for the trapezoidal diagram of Fig. 3 may be noted:

Strength of Concrete (psi)		2000	4000	6000
Energy per Unit Volume (in. lb./in^3)	From Triangular Part of Diagram	0.67	2.00	4.00
	From Rectangular Part of Diagram	5.33	4.00	3.56
	From Entire Trapezoid	6.00	6.00	7.56
Energy in 6 x 12-in. cylinder (in. lb.)	From Triangular Part of Diagram	226	678	1356
	From Rectangular Part of Diagram	1808	1356	1207
	From Entire Trapezoid	2034	2034	2563

These values follow very simply from the application of Equations (1), (2) and (3).

The author is in agreement with the comments made by Mr. Parsons with respect to the relationship of the paper to routine design. It may be mentioned that the criterion for the ultimate load on a beam, namely that the ultimate load is reached when the strain in the extreme fiber attains a value of ϵ_1, given by Eq. (3), may be expected to give good agreement with observed values of maximum load but not necessarily good agreement with observed values of maximum strain. This is due to the fact that the load-strain diagram at ultimate resistance of the beam is necessarily horizontal. The maximum load reached in the test is definite but the strain in the extreme fiber at this maximum load may be taken within wide limits.

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NOTE—For the convenience of the reader and to avoid duplications the author's original list of references (1 to 6) is relisted (with corrections) supplemented by the *additional* references in the contributions of the discussion—these additional references in a continuing series—7 to 20.—EDITOR.

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Electric Prestressing of Reinforcing Steel*

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SYNOPSIS

In the electric method of prestressing, smooth reinforcing rods are stressed by being temporarily expanded by an electric current after the concrete has hardened. The rods are coated with a thermoplastic material such as sulfur which melts and relieves the bond only while the rods are temporarily heated by the electric current. While the rods are expanded, nuts are taken up at the ends a predetermined amount to provide the desired prestress. Two notable features of the method are (1) that bond is restored with cooling and resolidification of the sulfur coating and (2) that the quick heating of the rods does not warm the concrete appreciably. Interesting results of tests and trials are reported.

The main purposes of prestressing are (1) to save reinforcing by permitting higher unit stresses and (2) to overcome shrinkage in the concrete. There are, however, other benefits which are not immediately recognized. One such benefit is the increased shear strength of prestressed concrete. A solid body in a state of tension can withstand very little shearing stress. Nearly everyone can recall using a knife in cutting a branch from a tree; one bends the branch and cuts it on the tension side, easily. Conversely compressive stress makes shearing more difficult. In all bodies except those whose plasticity borders on fluidity, compression across the surface of expected shear failure causes greatly increased strength. One of the remarkable features of the prestressed beam is the fact that the entire concrete section helps to carry the load. In the ordinary reinforced beam only the top third or so of the concrete is capable of carrying direct stress—all the remaining concrete is assumed cracked, throwing the entire remaining burden on the reinforcing steel. In the prestressed beam the entire concrete section, and especially the lower fibers, has an initial compression. Under bending load, these bottom fibers tend to lengthen, but due to their initial shortening from the pre-

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stressing they are still in compression and actually carry less stress than before loading.

In the electric method of prestressing reinforcing steel, the concrete is permitted to harden fully before the stress is applied, and bond to the steel is restored after the stressing. Smooth bars of reinforcing are used and these are coated before use with some thermoplastic material such as sulfur or low-melting alloys. An electric current of low voltage but high amperage is then passed through the bars. When the steel heats and elongates, nuts on the portruding ends are adjusted against heavy plate washers to obtain the amount of prestress desired. As the steel cools, the stress develops without further slippage in the concrete and the bond is restored by resolidification of the coating.

The application of the sulfur to the steel can be made in several ways, such as dipping in molten sulfur. It has been found advantageous to use a fibrous binder such as asbestos in the sulfur. The alternative coating with a low-melting alloy can also be applied in different ways, somewhat similar to tinning applications. The steel must be reasonably smooth. Round bars are preferred. The rods have threaded ends which extend beyond the boundary of the concrete or into specially molded depressions. The concrete surrounding the coated rods is permitted to harden and obtain full working strength before the prestressing is applied. This process, therefore, is not really prestressing in the sense of being applied prior to hardening of the concrete, but rather in the sense of being applied prior to the concrete being placed in service.

Alternating current is preferred for the quick heating and expansion of the rods because proper voltage and amperage can be obtained conveniently. The intensity of the current must be sufficient to expand the rods in such short time as not to heat the concrete appreciably. Satisfactory results have been obtained when the current has been 2500 amperes per sq. in. of steel section and limited to about two minutes. As previously stated, the rise in temperature causes the rods to elongate so that nuts on the protruding ends can be adjusted against heavy plate washers to obtain the amount of prestress desired. Thus, the stress develops upon cooling, while the rods remain fixed in length. When very long rods are used, separate sections can be connected when elongated, in order to eliminate the intermediate, plate washers. The electric method of heating and melting of the coating, which temporarily eliminates friction, causes a uniform elongation and therefore a subsequent stress which is uniform along the rods. The cooling after the current is stopped causes the coating to harden, and the bond between the rods and the concrete to be re-established. An important feature is the devel-

opment of bond *strength*, but not bond *stress*, during the cooling of the rods. The rods do not tend to slip through the concrete during the cooling, but the steel will withstand bond stress thereafter.

The desired adjustment or "take up" to be applied at the end of a rod immediately after the elastic heating, is the sum of three items; (1) elastic extension of the rod for the intended stress in the steel, (2) the resultant compression of the concrete, and (3) the expected shrinkage of the concrete.

The elastic extension of the steel rod is merely the product of the length and the strain. The strain is obtained simply from dividing the intended stress by the modulus of elasticity. For a 20-foot length and a desired stress of 20,000 psi, the "take up" for this item would be 20.20,000/30,-000,000, or 0.012 feet, somewhat more than an eighth of an inch.

The amount of "take up" to be added to provide for elastic compression of the concrete is similar to that for elastic extension of the steel but usually smaller and less certain. The stress in this case must be enough to balance the tension in the steel, and the strain is the stress divided by the modulus of elasticity. It appears most convenient to take account of plastic flow by using the "sustained" modulus of elasticity. For a wide variety of applications, it is believed sufficiently accurate to use as a "sustained" modulus of elasticity one-half of the ordinary or "instantaneous" modulus of elasticity. In cases of severe drying or of high and long-continued stress, an even lower value becomes appropriate. When the lower E is used to allow for plastic flow, it is apparent that the prestress when first applied will be slightly higher than the intended value but will gradually diminish toward the intended value.

Just as allowance is made for concrete compression by a moderate increase in the "take up" at the ends of the rods, so also is allowance made for expected shrinkage of the concrete. A distinction is made between so-called shrinkage in massive sections of concrete and drying shrinkage in relatively thin sections. In general, the only volume change in massive sections is attributed to change in temperature, and since steel and concrete have roughly the same thermal expansion and contraction no allowance need be made for temperature movements in the prestressing. The ultimate shrinkage of concrete in relatively thin sections due to long continued exposure to dry air may be as great as 0.06 per cent of the length. If such an extreme shrinkage is expected, a "take-up" of 0.012 ft. (0.14 in.) in a 20-ft. length would need to be provided for the shrinkage alone. This would mean an extra prestress in the steel of 20,000 psi. But this extreme shrinkage is outside the range of .015 to .045 per cent as given by the Joint Committee Report. A more common

degree of shrinkage for thin sections of concrete is about one-half of the maximum possible, or about 0.03 per cent of the length. It is apparent that only a moderate increase in the "take up" at the ends of the rods is necessary to provide for the usual shrinkage.

The prestress may be applied after partial shrinkage has occurred. By this expedient a lesser amount of the useful strength of the steel need be used for overcoming shrinkage.

TESTS OF ELECTRIC PRESTRESSING

Trial of feasibility of prestressing

One of the earliest trials of electric prestressing applied to a concrete beam was made with the assistance of the General Electric Co. The details of the beam: 20 ft. long, 20 in. wide, 6 in. thick; the reinforcement, 6 plain round 1-in. rods, (3 near each face with 1-in. coverage); the coating on reinforcement, a thin layer of sulfur applied by dipping.

A current of 2000 amperes was applied to the rods by using approximately 1.5 volts per foot of length. The expected lengthening of the bars occurred in about 2 minutes. The time required to melt the sulfur was found to be about one minute, by direct observation into a small recess extending to the steel. The heating and resulting extension of the rods was sufficient for a "take up" of $\frac{3}{16}$-in. at each end.

The concrete was not heated appreciably by the heating of the steel. The deflection of the beam was not more than $\frac{1}{8}$-in. and the heating was barely perceptible to the touch where the cover of concrete was $1\frac{1}{2}$ in.

A careful check of the faces of the beam showed no shrinkage cracks to have developed either due to setting and hardening or due to the prestressing procedure. William F. Lockhardt who wrote a report on this test on Sept. 30, 1942 states that the surfaces of the concrete were moistened in an attempt to reveal any fine cracks or checking, but none was found.

Bond stress after prestressing

The re-establishment of bond between the steel rods and the concrete was investigated by Prof. H. C. Berry of the University of Pennsylvania. A long beam of prestressed concrete was cross cut with a carborundum disc to produce specimen blocks of concrete including sections of the embedded and highly-stressed reinforcing rods, which had been treated as described. Four specimens were cut to 6-in. length and four to 12-in. The bond was tested by standing the specimens on thin strips of metal on a large spherical bearing block and pushing the rods through into a hole below. The pushing speed was 0.03 in. per minute and the beam was kept in balance during application of load. The behavior of the bond after first slip was of some interest. At the maximum load, the

beam dropped and the load fell back to about two-thirds of the maximum. Then the load built up slightly and slight slip occurred, with a click-like sound. There followed fairly regular slips and clicks, without much change in load until the rod had been pushed through about $\frac{1}{16}$ in. Thereafter the load fell off to about one-quarter of the maximum and the test was terminated when the movement was $\frac{1}{4}$ in.

The results of the push-out tests were as follows:

Specimen Number	Length of Embedment, In.	Maximum Load	Maximum Bond Stress, psi
1-1-6	6	4720	248
1-2-6	6	5020	265
2-1-6	6	5000	264
2-2-6	6	5320	281
	Average for 6-in. embedment		264
1-1-12	12	11800	314
1-2-12	12	15740	417
2 1-12	12	15040	399
2-2-12	12	17310	460
	Average for 12-in. embedment		397

It may be noted that the average bond strength was considerably greater for the 12-in. embedment than for the 6-in. This was attributed to a destruction of the bond for some distance from either end due to the cutting and the relief of stress at each cut. It is interesting to note that if the cutting were assumed to destroy the bond completely for a depth of 1.47 in. from either end, the bond strength would be 525 psi for both lengths of embedment.

Tests of resistance to fire

The low melting point which is a requisite of the coating material makes it apparent that the coating will melt and destroy the bond when sustained heating is applied to a structure where this type of prestressing has been used. A careful consideration of the conditions may reveal that this temporary lack of bond is not unduly serious. The failure of slabs and beams with conventional reinforcement is generally preceded by elongation of the steel and consequent tensile stress in the concrete leading to shear failure. Only those parts of a slab or beam which are compressed are capable of withstanding much shear stress.

The advantage of maintaining compression in the concrete by prestressing appears to be proved by a fire test which was made on the slab shown with a specially built furnace in Fig. 1. The slab was subjected to a four-hour fire in the special furnace, in which the temperature aver-

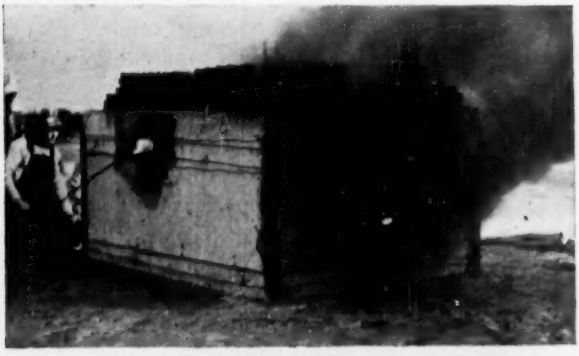

Fig. 1—Fire and load test on prestressed slab at Hooker's Point, Tampa, Fla. (Furnace fired with wood soaked with kerosene. Average temperature 1900 F. for last two hours.)

Fig. 2—Edge of slab carrying 400 pounds per square foot after fire test

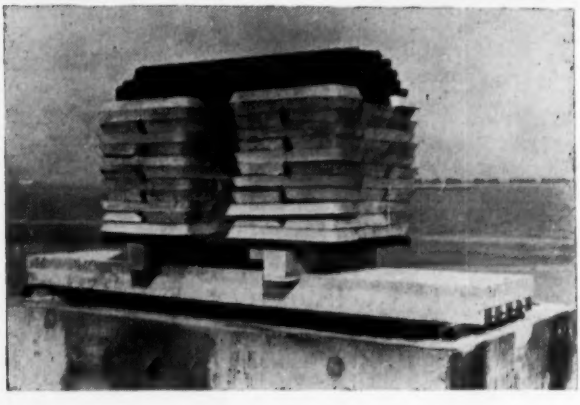

Fig. 3—Electric-prestressed slab carrying 6000 pounds of applied load

aged 1900 F. for the last two hours. During this test the slab carried a live load of 115 lb. per sq. ft. The sulfur coating on the rods melted during the first hour. After the fire was extinguished by sudden water cooling, the slab was loaded to 400 lb. per sq. ft. A close-up of the slab at this time is shown in Fig. 2. Although this load presumably stressed the steel to its elastic limit, no failure occurred.

Behavior of prestressed slabs under live load

The effect upon stresses in reinforcing rods due to added load was observed in two identical slabs with electric-stressed reinforcing. One of these slabs is shown under load in Fig. 3. The slabs were 7 ft. long, 20 in. wide, and 5 in. deep. Each was reinforecd with 5 longitudinal rods of $\frac{3}{8}$-in. diameter, one inch from the bottom face.

The slabs were made with 5.4 bags of high early strength cement per cubic yard. The aggregate was crushed limestone and the slump was 4 in. The slabs were kept moist at about 60 F. for 3 days, then allowed to shrink at the prevailing humidity of about 50 per cent for 2 more days. The sulfur-coated rods were given the electric prestressing at the age of 5 days, when companion tests showed the concrete to have attained a compression strength of 3000 psi. The "take up" on the rods during the electric heating was $\frac{1}{8}$ in., an amount computed to prestress the steel to 32,100 psi, assuming a sustained modulus of elasticity of 1,500,000 psi. for the concrete. Strain measurements on the steel rods with a 20-in. Berry strain gage revealed an average strain due to the prestressing of 1100 millionths. Assuming an "E" of 30,000,000 for the steel, the corresponding stress was 33,000 psi, which agreed very well with the intended value.

Load was applied to the slabs when they were 10 days old. In the meantime, shrinkage of the concrete had reduced the stress in the steel by an average amount of 7 per cent (75 millionths reduction in strain). Strains were measured with each 1000-lb. increment of total load, except at the highest load of 6000 lb., when it was deemed unsafe to work under the slabs.

The tensile stress in the steel increased only 14 per cent (4800 psi) due to the addition of 5000 lb. of live load. The total stress in the steel for the particular degree of prestressing applied, happened to be exactly the same as that computed for a similar, unrpestressed slab with the 5000 lb. load. Thus, the increase in stress due to the application of the load was less than 14 per cent of the corresponding stress in the unprestressed slab, using accepted methods of computation for the latter. At loads above 5000 lb., the total stress in the prestressed slab was less than that computed for a similar, unprestressed slab. The prestressed slabs behaved

elastically; strains increased regularly with increasing load, and no cracks were observed.

Deflections were measured with a steel scale to the nearest 0.01 inch. The upward deflection of the centers of the slabs due to the prestressing was found to be 0.04 in. Each 1000 lb. of applied load caused a downward deflection of approximately 0.01 in. At 5000 lb. of total load, the net downward deflection was 0.01 in.

The notable feature of the load tests on the two slabs was their rational elastic behavior. This was attributed to the absence of cracks and freedom from stress reversals.

ACKNOWLEDGMENT

The writers appreciate the interest shown in electric prestressing by many engineers and are especially indebted to the following: Chester L. Post, for contribution to stress analysis, Prof. H. C. Berry, for making bond tests, H. J. McGillivray for supervising fire test, T. J. Reading for making tests on loaded slabs, and General Electric Co. for investigations regarding apparatus and power requirements.

Discussion of this paper should be submitted in triplicate to the ACI Secretary before September 1, 1943, for publication in the JOURNAL Supplement concluding Proceedings V. 39.

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Discussion of a paper by Karl P. Billner and Roy W. Carlson

Electric Prestressing of Reinforcing Steel*

DISCUSSION BY P. W. ABELES and AUTHORS

By PAUL W. ABELES†

The method described by the authors constitutes a great improvement in prestressing. The authors say: "This process, therefore, is not really pre-stressing in the sense of being applied prior to hardening of the concrete, but rather in the sense of being applied prior to the concrete being placed in service."

The writer would like to point out that an application of tensioning to the reinforcement and transmitting compression to the concrete before the structure is "placed in service" relates to "prestressing" in general. The two different processes where the stretching process is carried out either before or after hardening of the concrete may be called "pre-stretching" or "post-stretching" respectively. In the first case, where the reinforcement is bonded to the concrete, the initial prestress is reduced at the release owing to the elastic deformation of the concrete and to shrinkage. In the second case, where the bond between reinforcement and concrete is normally destroyed. the compression is transmitted directly to the hardened concrete at the stretching of the reinforcement without any losses of the initial prestress occurring at this stage.

There is moreover a distinct difference in the behavior of prestressed concrete structures with bonded reinforcement and that in which the bond is destroyed. Tests have proved that in the latter case the deformations are much greater than in structures with bonded reinforcement.

The method with restoring the bond, described by the authors, represents, therefore a combination of the advantages of "post-stretching" and "pre-stretching" i.e., direct transmission of the compression to the hardened concrete, no immediate losses of the prestress and relatively small deformations of the structure, ensured by the bond between rein-

*ACI JOURNAL, June 1943; *Proceedings* V. 39, p. 585.
†Technical Advisor, London.

forcement and concrete. It would be interesting to learn the authors' view whether in their opinion this method is applicable also to thin wires. This appears doubtful in view of the small sizes which would not seem to allow a thread to be cut without a considerable reduction of the effective area. From the papers it is seen that the initial prestress amounted to about 30,000 psi. and the method was applied to rods. The temperature required to effect the elongation of 1/1000 corresponding to the ratio of the initial prestress to the Modulus of Elasticity amounts to 80 C. only. However, if steel of the highest possible strength at disposal is used, the initial prestress may amount to 200,000 psi and even more. In such a case the temperature necessary to effect the extension has to be very high (over 500 C.) and it may be expected that the strength properties would be affected and greatly reduced by such a treatment. This might influence the applicability of this new and very ingenious method and restrict its use to rods which are not tensioned higher than, say, 100,000 psi corresponding to a temperature of about 270 C.; but, of course, it will depend on results of special investigations to find out for each kind of steel that temperature which allows this method to be applied without causing considerable loss of strength.

AUTHOR'S CLOSURE

The authors appreciate the clear analysis of the method of electric prestressing by Mr. Abeles. In reply to the question of applying the method to thin wires, it is not likely that the method could be adapted readily to rods much less than ¼-in. in diameter. Our investigations indicate that it will be feasible to use rods from ¼-in. to 1-in. in diameter and up to 300 ft. long, but full benefit of high-strength thin wires would require special anchorages such as upsetting of the ends or affixing nuts without threads by welding or by hammering the ends to fill a conical hole through each nut. Research scientists of one of the best known steel manufacturers have informed us that high strength steel, whether it be regular cold drawn wire with high carbon content or one of the special alloy steels of very high yield point, generally start to loose strength when the temperature reaches 460 C. Therefore the limit of electric prestressing appears to be 165,000 psi.

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Job Problems and Practice

"JPP" is a means toward realizing more fully than in a limited number of longer JOURNAL contributions, the mutuality of ACI Membership effort to do a better concrete job. In JPP many Members may participate in few pages. So, if you have a question, ask it. If an answer is of likely general interest, it will be briefed here (with authorship credit unless the contributor prefers not). But don't wait for a question. If you know of a concrete problem solved—in field, laboratory, factory, or office—or if you are moved to constructive comment or criticism, obey the impulse; jot it down for JPP. Remember these pages are for informal and sometimes tentative fragments—not the "copper-riveted" conclusiveness of formal treatises. "Answers" to questions do not carry ACI authority; they represent the efforts of Members to add their bits to the sum of ACI Member knowledge of concrete "know-how."

Paint and Waterproofing Applied to Concrete Ships (39-133)

Q—A member asks information on painting concrete, particularly concrete ships, another correspondent asks about the application of waterproofing.

A—There is very little published information on the subject of painting concrete, except that contained in manufacturers' literature and advertising. Two brief discussions of experiences in painting concrete ships during the last World War will be found in the following references:

"Method of Construction of Concrete Ships" by R. J. Wig, Head, Concrete Ship Section, Emergency Fleet Corporation, ACI *Proceedings*, V. 15, p. 272.

"Concrete Ships Constructed by U. S. Shipping Board" by Walter R. Harper, Concrete Engineer, U. S. Shipping Board, ACI *Proceedings*, V. 18, p. 100.

A party to investigations on painting concrete ships conducted for the Emergency Fleet Corporation during the last war reports that the performance of the paints applied to the outside of the hull was very unsatisfactory. The principal difficulty was with poor adhesion and after a few months exposure to service, the paints tended to peal off in large strips. These paints were principally of the bituminous base type. Fairly satis-

factory results were obtained with spar varnish applied to locations not subjected to a severe moisture condition, such as the bulkheads, underside of the deck, etc.

Considering the exposure conditions, it would appear essential that the paint be of a type which is resistant to alkalies and will bond satisfactorily to damp concrete and will not tend to peal or lift as the result of a more or less continuously damp subsurface or to the tendencies of excess moisture in the concrete to escape through the painted surface. The paint should be of a type that has been proved satisfactory for use on other types of concrete construction subjected to dampness such as swimming pools, water tanks, etc.

There is considerable competent opinion that it is not necessary to paint concrete ships for the purpose of protecting the concrete and that the impermeable types of paint films are not only unnecessary but probably will be less satisfactory than those of the breathing type.

ACI Committee 616, Recommended Practice for the Application of Paint to Concrete Surfaces, R. E. Copeland, chairman, has the assignment comprehended in its title. It reported on portland cement paints*, but is yet to report on paints of other types.

As to waterproofing, an ACI member says it has not been found necessary to apply any waterproofing to the surface of those concrete ships which have been built, and further: "In concrete exposed to salt water action, however, the concrete should be dense and all reinforcement should be kept at least an inch from the surface and completely covered by the concrete, with no voids. The best modern practice is to use moderate heat portland cement, manufactured under Emergency Federal Specification E-SS-C-206a, or American Society for Testing Materials' Emergency Specification EA-C150-Type II.

"A paper describing the concrete pontoons or barges which were originally fastened together to form Lake Washington Bridge should prove of interest.† In the pontoons 'watertightness was achieved to a degree exceeding expectations,' with leakage (at time paper was written) nonexistent."

The Column Formula (39-134)

A building commissioner interested in revising a municipal building code asked for an explanation of the column formula in the ACI Code (ACI 318-41). Referred to a competent authority, here is the answer:

Formula 20 was based on a long series of tests sponsored by the Institute between 1930 and 1933. Some 600 column tests were made at Lehigh

*ACI JOURNAL, June 1942; *Proceedings* V. 38, p. 485.
†"Problems Presented by the Lake Washington Floating Bridge," by Charles E. Andrew, ACI JOURNAL, Jan. 1941; *Proceedings* V. 37, p. 253.

University and at the University of Illinois. Much of this information is included in ACI JOURNALS of that period and in Bulletin 267, University of Illinois. The form of this equation is based on the ultimate strength of the column, tests having shown that the value of n has very little bearing on the ultimate strength. The quantity n may have some significance at low stresses in the column, but as the loads become large the strength of the column depends entirely on the strength properties of the concrete and steel used. This is explained at length in the literature mentioned.

The use of the gross area of the column is also based on long study. Previous to the present Code regulations, the gross area was used on tied columns and the net section was used for spiral columns. This seemed to be entirely inconsistent. As a matter of fact, the gross area for spiral columns in the present regulations is dependent on the use of sufficient spiral reinforcement to produce at least as much strength, and a little more, than the strength attributed to the shell area of the column. Hence, in this type of column there are two lines of defense; the shell area, and, if loads reach a point where the outer shell of the column is crushed and spalls off, the spiral reinforcement to produce a little additional strength. This formula is based on the thought that the spiral should be used to produce toughness and resistance to sudden failure, but that when the shell of the column has spalled off there is not much need in developing much more strength in the column. As a matter of fact, the cases in which the shell of a column is destroyed in fire represent a very small fraction of the columns in service, and experience has shown that a spirally reinforced column, particularly a round one, is much more resistant to fire than the usual tied column.

Shear in Narrow Deep Beams (39-135)

Q—We have encountered a problem of very narrow deep beams, in which the depth nearly equals or exceeds the span. I have studied the paper by Molke and Kalinka, "Principles of Concrete Shell Dome Design,"* and an abstract of a paper by Dischinger, but need more light on the subject.

A†—Reference is made to "Concrete Walls for Earthquake Bracing of Tall Buildings."‡ Tests made at the University of Illinois on short, deep beams, reported in Bulletin 166, indicate a shearing stress of 800 psi for a length-depth ratio of 4.5. Tests made by Slater, Technical Bulletin 314, for a length-depth ratio of 3.3 showed shearing stresses of 1,340 to 1,963 psi. for shear reinforcement of .8 to 3.2 per cent.

*ACI JOURNAL, May-June, 1938; *Proceedings* V. 34, p. 649.
†By an ACI member.
‡*Engineering News-Record*, Sept. 3, 1931, p. 364.

From Slater's letter to the Editor of *Engineering News-Record*, in reference to the above article, in Nov. 5, 1931, issue, p. 741, we have:

The writer would not hesitate to design a deep girder for an ultimate load which would cause shearing stresses equal to one-half the compressive strength of concrete if vertical stirrups are used, or more, if inclined reinforcement is used. Such a girder would probably be short in proportion to its depth and would probably be an I-Section. This is not because of any limitation of the high shearing strength to cases of low length-depth ratios or to I-Sections, but because with such high shearing stresses it would be impossible to get sufficient longitudinal reinforcement into such a girder to resist the longitudinal stresses if the length-depth ratio is large and if the flanges are not made wider than the web thickness.

The paper by Dischinger, reference 26 of the article on Concrete Shell Dome Design appeared in Vol. 1, Publications of the International Association for Bridge and Structural Engineering. This paper did not cover "shear" but made a promise to do so at some later date. This paper did show a vast difference in moments obtained by his method as compared to conventional design. The moments were as much as four times greater than conventional moments in some cases. Another reference:

"Uber den Spannungs'ustand in Hohen Trägern und die Bewehrungwon Eisenbeton-Wäden." Verlag Conrad Witmer, Stuttgart, 1931.

Without going into mathematics there is a consideration in concrete design which will throw some light on this problem. Unit shear v as used in design is a measure of diagonal tension, but diagonal tension $t = \frac{1}{2}(s + \sqrt{s^2 + 4v^2})$ where s is the longitudinal tension at the point in question. Now safe value of v for use in design is based upon an assumption that t may be three times v. The maximum t would occur under a load in a beam loaded at the third points. At supports or at any point in short beams, the effect of s would be negligible and t would be, for all practical purposes, equal to v; v is pure shear and as the ultimate strength of concrete in pure shear is .50 to .70 f_c', short thin beams should withstand shearing stresses approximating .50 f_c'.

Vibration of Concrete in Test Cylinder Molds (39-136)

Q—What is the best current practice for the consolidation of concrete in test cylinder molds by means of mechanical vibration?

A—The Bureau of Reclamation *Concrete Manual* says (Fourth edition, p. 443):

For 6 by 12-in. and 8 by 16-in. cylinders the fresh concrete shall be placed in the mold in two layers, each approximately one-half the volume of the mold. Each layer shall be compacted by vibrating 20 seconds with an immersion type vibrator of appropriate size, penetrating only the layer to be compacted. In compacting the bottom layer, the vibrator shall not be allowed to rest on the bottom of the mold. In placing each scoopful of concrete, the scoop shall be moved around the top edge of the mold as the concrete slides from it, in order to insure symmetrical distribution of concrete in the mold. In

placing the concrete for the top layer, the mold shall not be filled to the extent that there will be mortar loss during vibration. After the vibration of the second layer, enough concrete shall be added to bring the level above the top of the mold. The specimen shall then be removed to the curing room and the excess concrete struck off with a trowel.

In the Bureau laboratories a 1-in. or 1⅛ in. rigid type, electric vibrator is normally used for 6 x 12 and 8 x 16-in. cylinders, and a two-inch flexible shaft ("wiggle-tail") machine is used for cylinders larger than 8 in. in diameter.

Advantages of Stockpiling Aggregates (39-137)

Q—What are the advantages and disadvantages of aggregate stock-piled in advance as compared with aggregate used as produced?*

By STANTON WALKER†

A—The most obvious advantage that comes to mind has to do with the greater uniformity of moisture content in stock-piled material as compared with material which has come directly from the plant. This advantage is brought more forcefully to our attention in connection with ready-mixed concrete operations than with job-mixing operations. In the usual job operation aggregates often are stock piled as a matter of convenience in handling and, therefore, the comparative advantages and disadvantages are not emphasized. In many ready-mixed concrete operations the central-mixer or the transit-mixer is charged directly from hoppers which have been loaded directly from the washing plant. In the absence of an opportunity for drainage the moisture content of fine aggregate may vary from a maximum of 17 or 18 per cent down to 4 or 5 per cent. With such fluctuation in the moisture condition of the aggregate, it is extremely difficult to control the proportions of the concrete with any degree of accuracy.

As a result of such variations more and more specifications require aggregates to be stock piled for some stated period before use. That is not a complete solution to the problem since it does rain now and then. Moreover, different materials drain at different rates. I would think a more logical requirement would be one that the material contain moisture less than some stated amount, or be controlled within stated limits, and then let the ready-mixed concrete operator or the contractor determine how to obtain that condition.

There is a special application of stock piling that may be extremely important in the case of certain aggregates and in certain localities. Some tests showing the effect of moisture content in the aggregates on

*Quiz Session, ACI Convention, 1942.
†Director of Engineering, National Sand and Gravel Association.

the ability of concrete to resist freezing and thawing, impress us with the much longer life of concrete into which these aggregates were introduced dry as compared with that made with the same aggregates which were saturated when mixed in the concrete. The relationship would not apply, I believe to all materials.

It is possible that stock piling of materials of the nature used in the tests referred to would serve to increase their durability very considerably when exposed to environments in which they contribute to the disintegration of the concrete.

One last word; I do not want to "promote" the stock-piling idea; it costs money.

(The foregoing prompted an inquiry from the floor: "How effective is our attempt to maintain a definite water-cement ratio. where we add extra water at the time of mixing, to compensate for the amount of water yet to be absorbed by the aggregate, particularly in view of the time available for absorption in the wet mix, and the rate at which aggregates absorb water," discussed as follows by Mr. Walker.)

Some aggregates become completely saturated only after a very long time. We have kept certain materials, with a fairly high absorption, immersed in water for approximately three years and, within that period, they have absorbed about 90 to 95 per cent of the amount of water for which pore space is available, and of the amount which they had in them when dredged from the river. The absorption of aggregates is commonly determined after immersion in the water at room temperature for 24 hours. In the case of concrete we might assume two or three hours as being available for the absorption of water while the concrete is in a plastic state. As a general average I would say that aggregates will absorb in a period of two or three hours about 80 to 85 per cent of the amount absorbed in 24 hours.

By T. C. POWERS*

One aspect of this question has not been emphasized: In correcting the total water content, so as to obtain the net water-cement ratio, one should keep in mind that he is attempting to determine the amount of water that occupies space in the concrete, or the amount of space in the concrete that is or could be occupied by water. If you accept that premise, then the amount of water that may be absorbed after the concrete has become rigid should not be subtracted from the total. It might be that a dry aggregate could absorb water for twenty-four hours at an appreciable rate, but the concrete cannot shrink in volume for twenty-four hours. When the aggregate is bone-dry, and when it is known to absorb water for an appreciable time, say five or ten hours, there is no way of calculating accurately what the net water-cement ratio is.

*Portland Cement Assn., Chicago.

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Current Reviews

of Significant Contributions in Foreign and Domestic Publications, prepared by the Institute's Reviewers

Static water tanks

J. T. GRIFFITH, *The Structural Engineer*, V. 21, No. 1 (Jan. 1943) pp. 1-11.

Description is given of the structural features of water tanks to be used as a source of static water in industrial towns when water mains have been rendered useless due to air raids. A town is divided into High and Low Risk Areas with appropriate allotments of water supplied by tanks placed at suitable locations. Two concrete tanks are described one having a capacity of 100,000 gallons and the other of 400,000 gallons. Construction recommendations and structural details are provided.

Beam-method of design of eccentrically-loaded columns

ODD ALBERT, *Engineering News-Record*, V. 130, No. 10 (Mar. 11, 1943), pp. 117-119
Reviewed by S. J. CHAMBERLIN

The eccentric load is considered as being replaced by an equivalent compressive load acting at the tension steel and a couple. The tension steel area may be computed as in a beam, using the couple as the moment. From the flexural steel area an area equal to the compressive load divided by the working steel stress is deducted. The compression steel area may be computed by treating the column as a beam. If the stresses are given and the dimensions sought, the "beam method" will give the results directly whether tension and compression steel is used or tension steel only. Illustrative examples are given.

Strength of steel anchors in concrete

HARRY E. GRAHAM, *Engineering News-Record*, V. 130, No. 16 (Apr. 22, 1943), pp. 80-81.
Reviewed by S. J. CHAMBERLIN

Round and square 1¼-in. bars were embedded in old concrete at distances varying from 10 to 40 in. and then pulled out under field conditions with a hydraulic jack. Conclusions based on 11 tests at 7 days with 1 to 1 grout-filled holes include: (1) beyond a certain minimum depth an increase in embedment will give little or no increase in strength; (2) a bent bar in a sloping hole will usually give a lower value than a horizontal bar; (3) for the same surface area deformed bars show considerably greater strength than plain bars. Compacted lead wool did not give satisfactory results for large loads. A table gives the resistance of the bars to withdrawal.

Precast units cut cost and time on seaplane ramps

Engineering News-Record, V. 130. No. 8 (Feb. 25, 1943) pp. 66-69. Reviewed by S. J. CHAMBERLIN

Precast beams and slabs are installed without cofferdams on previously driven piles. The beams are of H-section, 37½ in. deep with a 12 in. thick web, and boxes are cast

integral to engage the tops of the piles. Space is provided in the boxes for grouting and alignment of the beams. The slabs, 15 ft. square and 10 in. thick with side flanges 24 in. deep, fit into the slotted keyways on either side of the beams for support and are slid into position. No bolts, dowels or fastenings are necessary, the only underwater work required being the placement of grout in the boxes of the beams over the piles. In an earlier design, large "cap slabs" were supported at the corners on piles, and the various units tied together with dowels and grout.

Multiple-arch concrete roof saves steel

Engineering News-Record, V. 130, No. 2 (Jan. 14, 1943), pp. 99-102 Reviewed by S. J. CHAMBERLIN

The framing is characterized by multiple thin-section arch slabs spanning 38 ft. between concrete girders, which in turn, are supported by concrete columns every 30 ft. The arch slabs have a constant radius of 47 ft. and are 3 in. thick except at the edges where they meet the 18 by 27-in. girders. The slabs are reinforced with only a single layer of 4 x 4-in. No. 6 woven wire mesh. Each 30-ft. length of arch roof is stiffened by concrete ribs, 5¾ in. wide and of uniform thickness, located at the column lines and at the center of each bay. These ribs are solid except for holes for piping. All bays are similar except for specials and except for a section over a heat treatment pit, where 63-ft. spans are used. Longitudinal and transverse expansion joints of wood with asbestos flashing are provided. The design requires only 2.7 lb. of steel per sq. ft. of floor area.

Mortar-coated piles resist hard treatment

Engineering News-Record, V. 130, No. 12 (Mar. 25, 1943), pp. 73-75. Reviewed by S. J. CHAMBERLIN

Further experience of shotcrete protection for wood piles (reviewed in JOURNAL of Feb. 1943) indicates them to be resistant to rough handling and fast driving. The mortar-covered piles averaged 108 ft. in length with jacket lengths from 15 to 48 ft. Lengths up to 100 ft. were lifted and set in place by a single line. By increasing the lap of the wire mesh to about 6 in. and putting 4-in. square shear pockets at 18-in. centers instead of the 24 in. previously used, it was possible to drive the piles with 15,000 foot-pound blows at the rate of 120 per minute. One test pile was driven to practical refusal at this rate without developing cracking or breaking the coating except at the top 2-ft. length closest to the hammer. The mortar jacket increases the diameter and load-carrying capacity as well as stiffening the pile. The mortar-covered piles were considered cheaper and quicker than several other types used on the job.

Navy builds prestressed concrete tanks

R. B. GARRABRANT, *Engineering News-Record*, V. 130, No. 4 (Jan. 28, 1943), pp. 57-59.
Reviewed by S. J. CHAMBERLIN

The underground fuel tanks have been built up to 50,000 bbl. capacity. The floor slab, with which is combined the wall and roof column footings, are laid by conventional methods. Aroung the circumference of the tank floor the top surface is stepped down to provide a shelf for supporting the floor, and is painted with asphalt to break the bond between the wall and its footing. The wall is placed in segments to its full height and to about three-quarters of its final thickness. Sixty hours after the last wall section is placed the horizontal circumferential hoops are applied. The rods are threaded and connected by turnbuckles for prestressing. The tightening is gone through twice, progressing from the bottom to the top, with force applied simultaneously to all turnbuckles in a single loop. The degree of prestressing is determined by the moment on the turnbuckle. The steel and concrete for the columns and roof are placed after the wall steel has been fully prestressed and the side wall is then built up to full section using

either cast-in-place concrete or shotcrete over wire mesh. Special linings have been developed: sodium silicate for fuel oil; inert resin base enamel for diesel oil; resin base enamel or organic polysulfide film for aviation gasolines.

Concrete paving in near zero weather

Engineering News-Record, V. 130, No. 16 (Apr. 22, 1943), pp. 67-70 Reviewed by S. J. CHAMBERLIN

To meet wartime necessity, runways for a large airport (1½ million sq. yd. of concrete) in the Pacific Northwest were paved in subfreezing temperatures. Aggregate producing plants were enclosed to prevent freezing. Storage bins for aggregates were enclosed and steam heated. Water at near the freezing point from a nearby lake was heated to as high as 160 F in a housed plant. Insulated tank trucks transported the water to the mixers with a loss in temperature of 10 to 15 degrees. The cement content was increased from 1¼ to 1½ bbl. per cu. yd. and calcium chloride was added at the rate of 1 to 2 lb. per sack of cement depending upon the temperature. The subgrade was protected by blanketing it with a 6-in. layer of bank sand which was removed immediately preceding paving by carryalls or bulldozers after being bladed. Fine grading was delayed until the approach of the pavers. As soon as the newly finished concrete was sprayed with a liquid curing compound, hay was spread over the surface in layers about 1 ft. thick. On top of the hay a heavy paper covering was placed, which was sealed at the joints and weighted at the edges. Concrete at the average rate of 5,350 sq. yd. per day was placed as low as 7 F with nights as cold as −9 F.

Big concrete plant speeds power dam

Engineering News-Record, V. 129, No. 25 (Dec. 17, 1942) pp. 78-80. Reviewed by S. J. CHAMBERLIN

The plant for a concrete gravity dam 200 ft. high is capable of placing a maximum of 4,500 cu. yd. daily with an average of 2,000 cu. yd. Natural sand is obtained downstream and is delivered by truck to a belt conveyor where it can be blended with manufactured sand from crushed limestone. The 30 in. by 350-ft. belt discharges into a 40,000 cu. yd. storage area. Coarse aggregate in four separate sizes is delivered by truck to hoppers feeding belt conveyors leading to storage piles. Running under the sand storage and the four aggregate piles is a 9 by 9-ft. reclaiming tunnel constructed of wood. The 36-in. belt through the tunnel feeds two other belt conveyors, one leading to the main mixing plant and the other to a truck batcher plant. Aggregate storage for 300 cu. yd. of each size of aggregate and 275 cu. yd. of sand is provided in bins above the four 4-cu. yd. mixers. Bulk cement is delivered by truck to three steel silos at the plant. A bin above the mixer provides a storage of 600 bbl. Water is obtained from the river. The concrete is distributed in 4-cu. yd. buckets handled by a railroad to two hammer-head cranes operating on a trestle. A double-drum paver is used to place concrete for auxiliary work out of reach of the cranes. Cantilevered forms are being used, with the concrete placed in alternate blocks in 5-ft. lifts.

Industrial buildings of precast concrete

Engineering News-Record, V. 130, No. 8 (Feb. 25, 1943) pp. 62-65. Reviewed by S. J. CHAMBERLIN

Precast columns, girders, beams, wall panels, and floor and roof slabs are manufactured in an enclosed plant. A 1:1½:3 mixture with high-early-strength cement makes possible a strength of 5,000 psi in 48 hr. Concrete forms that may be used many times for columns and girders have sloping sides to permit easy removal. Columns and girders of many sizes are possible through the use of several sizes of forms and the use of fillers. The columns are cast with a flared base penetrated by four holes for 1-in. anchor bolts set in the 1-ft. thick by 5-ft. square footings. Steel pipes are welded to the girder reinforcement to provide holes for the bolts embedded in the columns. Similar holes are

provided in the ends of the beams for bolts that tie the framework together. Roof and floor slabs are 12 in. wide with depths of 3 to 4 in. The space between bolts and pipes, seatings and joints are grouted. The reinforcing of the girders is butt-welded over the columns and the reinforcing of the beams is welded at their ends. Advantages claimed over field-cast concrete are faster and better controlled placing, savings in forms, erection in cold weather is simplified and shrinkage occurs before erection. Advantages over other materials are that no additional fireproofing or protective coatings are necessary and early deliveries are assured.

Ten-year tests of high-early-strength cement concretes

LOUIS SHUMAN, Bureau of Standards. Journal of Research, V. 29, No. 6, Dec., 1942.
HIGHWAY RESEARCH ABSTRACTS

Ten-year compressive-strength tests have now been completed on concretes made with 12 high-early-strength cements, for which the results up to 1 year were given in a report in 1935 (R.P. 799). The concretes were of three different cement-water (C/W) ratios, five initial temperature conditions, and four curing conditions.

The concretes stored in damp air generally continued to gain strength up to 10 years. Concretes stored in the air of the laboratory had about the same strengths at 10 years as at 28 days. The strengths at ages after 28 days were not appreciably affected by the initial temperatures, which varied from 70 to 110 F. However, the usual variation in strength with C/W ratio persisted up to 10 years. For damp-cured 1:2:4 concrete of C/W ratio = 1.50 (7.5 gal. of water per 94 lb. bag of cement) the compressive strengths at 10 years were generally between 4,000 and 5,000 psi. For concrete of the same proportions, and a C/W ratio of 1.73 (6.5 gal. per bag), strengths of over 6,000 psi. were attained for 8 of the 12 cements.

Present-day cements, even of the moderate-heat type, are shown to be capable of giving concrete strengths in 1 month at least equal to those for a 1910 cement at 10 years; present-day (1941) high-early-strength cements may give strengths at 1 mo. exceeding the 10-year strengths.

The shape of concrete aggregate and its testing

E. SCHIEL: *Betonstrasse*, 1941, 16 (18), 181-6. Road Abstracts, 1943, V. X, No. 3, England.
HIGHWAY RESEARCH ABSTRACTS

To define the shape of aggregate it is desirable to determine the average shape of a given mixture and to sort the particles according to shape. By means of apparatus used in ore-dressing, it was found that different shapes could be distinguished by their settling velocities in water. An apparatus was therefore employed in which the particles were sorted according to the time taken to fall through 1 metre (39.4 in.) of water. It was observed that particles tend to fall in such a position that the greatest resistance is offered to the water, and that cubical particles fell fastest and flaky slowest. Tests indicated that for particles of the same specific gravity, settling velocity is a function of the thickness of the particle. Formulae are derived relating these three factors. A combination of this result with sieving, which gives the mean dimension (breadth) of particles, will therefore define the shape of the aggregate. In order to use these facts to appraise the quality of aggregate, a formula is developed that gives selected aggregate of good shape the value $f = 100$. The following values for f have been found for different shapes of aggregate: very flaky, < 80.5; flaky, between 80.5 and 83.5; fairly cubical, between 83.5 and 86.5; and cubical, > 86.5. The results of preliminary tests on concrete cubes made with aggregates of the same grading and of three different shapes showed that compressive strength and density were higher as aggregates with a higher settling velocity were used. Cubical aggregate also gave more workable mixes with the same quantity of water.

Significance of tests of concrete and concrete aggregates—1943
American Society for Testing Materials; 175 p., $1.50.

Sponsored by ASTM Committee C-9 on Concrete and Concrete Aggregates, with the cooperation of some 25 authorities, this latest report issued by the American Society for Testing Materials discusses the significance, limitations, and applicability of the tests standardized by the society, of some tests not yet standardized, and of a number of tests of a research nature not amenable to standardization. A list of the tests and the authors who prepared the materials for discussion follows:

Tests for concrete: Compressive strength, Lyse (Litehiser); Tensile and Flexural, Goldbeck; Shearing and Torsional, Richart; Elastic Properties, Teller; Durability, Scholer; Wear Resistance, Tuthill and Blanks; Workability, Powers; Volume Changes and Plastic Flow, Davis and Kelly; Cement Content, Kriege; Uniformity, Bertin; Permeability and Absorption, Withey; Density and Unit Weight, Swayze; Curing, Kellerman.

Tests for aggregates: Size, Shape, Surface Texture, and Grading, Gilkey; Specific Gravity, Unit Weight, and Voids, Hubbard; Soundness, Walker; Abrasion Tests, Clemmer; Free Moisture and Absorption, Myers; Deleterious Substances, Lang; Mortar Strength, Jackson; Mineral Composition, Kriege.

The book is completed by a general discussion on "The Numbers of Specimens or Tests of Concrete and Concrete Aggregates Required for Reasonable Accuracy of the Average," by R. W. Crum and H. W. Leavitt. The committee in charge, which correlated the work and arranged for approval by Committee C-9 was headed by R. W. Crum, serving with Messrs. Stanton Walker and A. T. Goldbeck.

Conservation of construction equipment and facilities
Associated General Contractors of America, Inc. HIGHWAY RESEARCH ABSTRACTS

This 80-page book has been published as an aid in the war effort, through conservation of critical materials and maintenance of equipment, now impossible to replace, in usable condition for any performance that may be demanded from it.

Of convenient pocket size, the book is designed for use by key men directly responsible for the operation and maintenance of equipment and critical materials.

Opening chapter of the book is entitled Plant Layout and presents suggestions for laying out a construction project so as to conserve manpower and economize in the use of critical materials. Construction Equipment is the title of the second section which presents basic principles governing use, care and repair of equipment.

Separate chapters are devoted to internal combustion engines, electric equipment, automobiles and trucks, small tools, rubber, wire rope and manila and hemp rope, tarpaulins and canvas, scaffolding and steel piling and forms.

Forty-three items of construction equipment are listed briefly in a "Glossary of Maintenance Hints," which gives specific points that require special attention and suggested methods of preventing and curing trouble.

Fire prevention and fire fighting are covered by a chapter of the manual and another chapter covers accident prevention. Sabatage precautions are outlined in one section.

The final chapter of the book is devoted to salvage and reclamation, which, the manual points out, should be a continuous activity to be vigorously pursued.

A reference list is included, giving the names of manufacturers of various kinds of equipment, from whom further literature and suggestions may be secured in connection with maintenance of their equipment.

Copies may be secured from the Construction Foundation, Munsey Building, Washington, D. C., at 50 cents each, $5.00 per dozen and $25.00 per hundred.

The development of lightweight concretes from North Carolina vermiculites

WILLIAM A. SCHALES, A. F. GREAVES-WALKER, E. R. TODD AND D. F. COX. Bulletin 24, Engineering Experiment Station, North Carolina State College of Agriculture and Engineering, Raleigh, N. C., May 1942.

The following is from "Conclusions and Recommendations" of the Bulletin:

Maximum expansion of North Carolina vermiculites is obtained at a temperature of 1840 F, applied suddenly and for a short time and maximum toughness below 1900 F. N. C. vermiculites so treated are suitable for light weight concretes having good resistance to fire, and useful for sound and heat insulators. They "may be cut with a wood saw or nailed without damaging or cracking the material.

"Vermiculite concretes are extremely porous, but can be water-proofed by stuccoing or by coating them with a 1:2 mix of cement and sand. Commercial concrete waterproofing agents are not effective in water proofing vermiculite concretes. Mixes of Lumnite cement and expanded vermiculite have a high resistance to thermal shock. High-early-strength cements may be used instead of standard portland cements without affecting the physical properties of vermiculite concretes.

"To obtain maximum expansion and the maximum amount of golden material, North Carolina vermiculites should be subjected suddenly to a high temperature (1840 F) under oxidizing conditions for a short period.

"For general construction purposes, the best concrete can be produced with a 1:4 mix using 4.24 gal. of water per cu. ft. of vermiculite and a mixing time of 5 minutes. The physical properties of this mix are as follows: compressive strength, 297.0 psi; modulus of rupture, 111.3 psi; yield per cubic foot of cement (bag), 3.9 cu. ft. of concrete; unit weight or density, 34.2 lb. per cu. ft.; conductivity, 1.10 BTU per hour per sq. ft. per degrees F per inch at 85.5 F.

"To be made waterproof, vermiculite concretes should be coated with a layer of dense concrete (1 part cement: 2 parts sand), the maximum required thickness being ⅛ in. For best results this coating should be treated with a waterproofing agent."

Apparatus for particle size analysis

C. J. STAIRMAND AND W. F. CAREY: *Engineering*, 1942, 154 (3997), 141-2; (3999), 181-2; (4001), 221-2; (4002), 241-2. Building Science Abstracts, V. XVI (New Series) No. 1, Jan. 1943. Apr., 1943.
HIGHWAY RESEARCH ABSTRACTS

A description is given of a new form of photographic sedimentation apparatus developed in the laboratories of Imperial Chemical Industries, Ltd., and the results of performance tests with this instrument and with other existing types of apparatus are compared. The principle of the method is to photograph the tracks of particles within the range 2 microns—100 microns as they fall in a stagnant liquid under gravitational attraction. The track length of each particle, for a certain exposure period, is a measure of its falling speed and by applying Stokes' Law the particle size may be calculated and a grading curve obtained. Light from a carbon arc lamp illuminates the sedimentation cell, 2 ft. high and 600 ml. volume, which holds a suspension of the powder in alcohol or ethylene glycol stirred by compressed air bubbles to ensure thorough mixing. Thermal convection currents are reduced by having vertical glass baffles in the cell and enclosing it in a water-jacket through which cold water circulates. By means of a projector an enlargement (x16) of the particle tracks obtained with a ¼ plate magazine type camera is thrown onto a screen. The complete optical and photographic system together with the method of carrying out a particle size analysis on a given sample are detailed. Calibration of the method showed that correct absolute values of size grading were obtainable, with random errors not affecting the result by more than a few per cent. This apparatus has also been tried out, as a reference standard, against other simpler snd cheaper forms of particle sizing apparatus, including elutriators (Roller, Gonell, Schone and Andrews' types) sedimenters (Werner and Andreasen types), turbidimeters, and the microscope

count. Several of these methods are here described, the sphere of usefulness of each being indicated as well as its peculiar advantages and limitations, and directions in which improvements might be made. The results have shown that the most suitable methods for particle-size analysis are: (1) For ranges of 0.3 to 10 microns microscopical examination with eyepiece graticule and a satisfactory optical system. (2) For ranges of 2 to 100 microns photographic sedimentation. (3) For ranges of 10 to 80 microns elutriators, bearing in mind the limitations of each type.

Building concrete ships in prefabricated forms

Engineering News-Record, V. 129, No. 27 (Dec. 31, 1942) pp. 30-34. Reviewed by S. J. CHAMBERLIN

The first of the 14,000-ton seagoing concrete barges of 56-ft. beam, 38-ft. depth and 375-ft. length, showed up well after form stripping and also under hydrostatic test. The shipyard is divided into four major sections: graving docks with outfitting pier, form yard, reinforcing steel yard, and batching plant. Provisions have been made to fabricate and assemble sections in the form and steel yards within the capacities of the cranes. Exterior form assemblies are built on an open "mold loft." The heaviest form sections are near the prow and stern weighing as much as 12½ tons. Amidships sections are simple, straight forms and are built out in the yard, not in the mold loft. Reinforcing bars, continuous for the full length of the ship, are made up of 80-ft. lengths flash welded together and pulled into the ship through a hole in the stern. Reinforcing for the ribs, for the transverse bulkheads and for the longitudinal bulkhead, is fully assembled in the steel yard. With outer forms at full shell height, when the steel in floor and frames is completely in place, the inside forms, up to elevation 4 ft. 5 in., are set in 18-in. steps as the first concrete casting is progressively made. After the first casting is completed and the inside forms stripped, the transverse bulkhead steel is set and preparations are made to place the inside forms from elevation 4 ft. 5 in. to deck level for the second casting. After the second casting, the deck steel goes in and the deck is placed. The concrete is delivered by crane and bucket to hoppers that charge hand buggies operating on runways. The light-weight concrete shell is 5 in. thick across the bottom and up around the turn of the bilge where it begins to decrease until it becomes 4½ in. at the 18-ft. level, which thickness extends to the deck. Vibrators are used against the forms, on the projecting bars and bayonet type vibrators are inserted in the concrete. Three coats of coal tar paint and a coat of copper sulphate anti-fouling compound is applied on the hull exterior below the water line. The interior concrete surface of the oil cargo tanks is treated with "Amercoat" and the interior of the ballast tanks with "inertol." Total steel requirement is about 60 per cent of a comparable vessel of conventional steel design. The first barge of 50,000-bbl. oad capacity contained 3,100 cu. yd. of concrete requiring 8,750 bbl. of cement.

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PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

VOLUME 39—1943

From JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, Vol. 14, Sept. 1942, to June 1943 and Supplement, November 1943.

This is an Index of:

Original contributions to the JOURNAL OF THE AMERICAN CONCRETE INSTITUTE—papers, reports, discussions by subject, title and author.

For the convenience of the reader who is referring to JOURNAL issues rather than to Bound Volumes (in which discussion is assembled following the paper discussed) there is a reference to JOURNAL issue in which discussion appeared. This reference to discussion is by supplementary page numbers. If the last page of a paper is 28, the first page of the discussion, published later, is 28 - 1.

In general, important subjects are classified and indexed under approximately 30 main headings each one appearing in its proper alphabetical order in bold face capital letters—as for instance, ARCHITECTURAL DESIGN. "Housing," and other subjects classified under this head, are indented.

Key words to important subjects, appear, in alphabetical order in addition to the general classification—as for instance "Admixtures" and "Aggregates" each referring to MATERIALS AND TESTS under which all allied references appear, indented. Authors' names and original titles of papers appear in bold face type in proper alphabetical sequence with the subjects, with references to their contributions.

Specific data on Beams are so indexed by reference to ENGINEERING DESIGN or TESTS OF MEMBERS AND COMPLETED STRUCTURES thus avoiding an oversight by the searcher of important allied data.

The complete title of each paper is indexed. This title is repeated in full following the name of the author in its proper alphabetical order. Numerous cross references to parts of the subject matter covered, do not always indicate the full scope. This may be found by looking up the item referred to under the name of the author.

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